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Geological-Seismological Evaluation of Earthquake Hazards at St. Stephen Powerhouse, Cooper River Rediversion Project, South Carolina, and Newmark-Sliding-Block Type Deformation Analysis of Embankments

by Ellis L. Krinitzsky, Mary E. Hynes, Donald E. Yule, Richard S. Olsen



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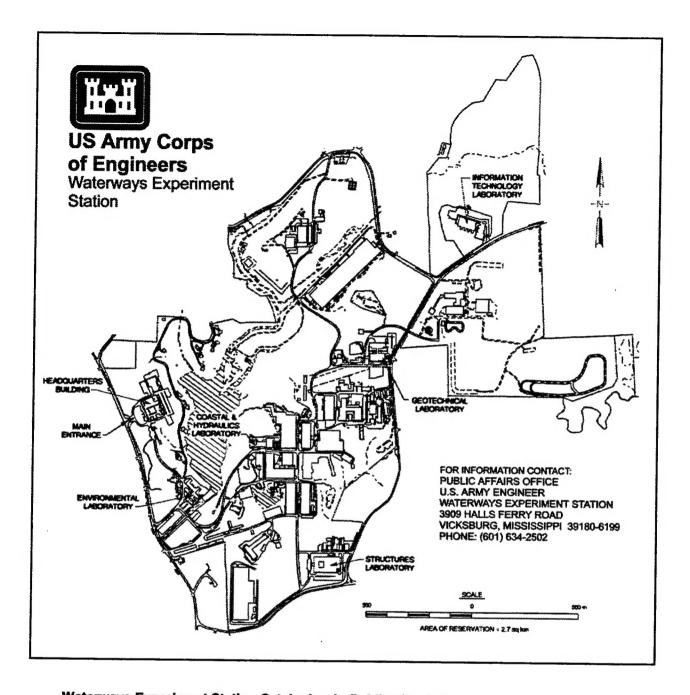
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CO)

Preface

This report summarizes a study conducted by the U.S. Army Engineer Waterways Experiment Station (WES) for the U.S. Army Engineer District, Charleston, SC (CESAC). The CESAC Project Manager was Mr. Wayne Bieganousky, Chief, Geotechnical, Materials, Sitework and Navigation Section (CESAC-EN-DF).

Dr. Ellis L. Krinitzsky, Geotechnical Laboratory (GL), and Mr. Donald E. Yule, Earthquake Engineering and Geophysics Branch (EEGB), Earthquake Engineering and Geosciences Division (EEGD), GL, conducted the portion of the study regarding seismic hazard. Dr. Mary E. Hynes, Chief, EEGB, Dr. Richard S. Olsen, EEGB, and Mr. Yule conducted the portion of the study regarding displacement analyses. Mr. Joseph B. Dunbar, Engineering Geology Branch (EGB), EEGD, GL, assisted the project considerably by collecting background information about the project, construction and design records, and regional geological and seismicity information.

Overall direction at WES was provided by Dr. Lillian D. Wakeley, Acting Chief, EEGD, and Dr. William F. Marcuson III, Director, GL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Robin R. Cababa, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	Ву	To Obtain
feet	0.3048	meters
inches	2.540	centimeters
miles (U.S. statute)	1.609344	kilometers
pounds	0.4535924	kilograms, assuming G=980.665 cm/sec ²
pounds	4.4481	Newtons
pounds (force) per square inch	175.1	Newtons per square meter
pounds (force) per square inch	6.8947	kiloPascals
tons per square foot	95.8	kiloPascals
atmospheric pressure	1.0332	kilograms per square centimeter, assuming G=980.665 cm/sec ²
atmospheric pressure	101.325	kiloPascals

Note: 1 atm = 14.696 psi = 1.0581 tsf = 1 ksc = 100 kPa

1 Introduction

At the request of the U.S. Army Engineer District, Charleston, the U.S. Army Engineer Waterways Experiment Station conducted an evaluation of the geological-seismological hazard at the St. Stephen Powerhouse Project, which is part of the Cooper River Rediversion Project in South Carolina. The project is located about 60 km north of Charleston, SC, and consists of a reinforced concrete powerhouse structure founded on rock, flanked by rolled-fill earth embankments, founded partially on rock and partially on alluvium. For the purposes of this study, the alluvium is assumed to be competent, not susceptible to liquefaction.

Executive Summary

The Maximum Credible Earthquake (MCE) is estimated to correspond to a magnitude 7.5 event, 55 km from the site, resulting in peak ground accelerations at the site of 0.32 to 0.35 g. The Operating Basis Earthquake (OBE) is estimated to correspond to about a magnitude 5 event, resulting in a peak ground acceleration of 0.04 to 0.05 g at the site. The Newmark-sliding-block analyses indicate deformations in the maximum section under the MCE will be negligible, less than 1 cm. However, deformations under retaining walls and embankments founded on natural ground may be on the order of 15 to 35 cm.

Purpose and Scope

The purpose and scope of this study are as follows:

- a. Determine rock outcrop ground motions appropriate for seismic analysis of embankment dam and reinforced concrete control structures, to include peak ground motion parameters, recommended analogous accelerograms, and response spectra.
- b. Provide these recommendations in a letter report, to include the basis for selection of these motions, historical seismicity of the area, identified seismic source zones and hot spots, and basis for attenuating these motions to the site.
- c. Since this is a low hazard dam with high consequences of failure, provide ground motions ranging from MCE motions to standard OBE motions

(corresponding to a return period of 144 years as recommended in ER 1110-2-1806).

- d. WES personnel visited the Charleston District to collect background information about the site and dam structures necessary for the selection of ground motions and as needed for a preliminary deformation analysis of the embankment structure.
- e. Conduct preliminary seismic response and deformation analysis of the embankment, and include in the report. It is assumed that sufficient information is available from design memoranda to estimate input parameters for the embankment deformation analysis.
 - f. An evaluation of liquefaction potential is beyond the scope of this study.

Organization of Report

The earthquake ground motions, ranging from MCE to OBE, are provided in Chapter 2 with the basis for selecting these motions. Chapter 3 contains the results of the Newmark-sliding-block analyses. References, Tables, and Figures follow the text. Appendix A contains a detailed listing of the seismic history of the project area.

2 Geological-Seismological Evaluation of Earthquake Hazard

Background

Purpose and Scope

The purpose of the geological-seismological investigation is to evaluate the earthquake hazards at the St. Stephen Powerhouse site. The objective is to provide ground motion parameters, response spectra, and analogous accelerograms for the earthquake ground motions that would be felt in the free field at the site. The ground motions defined by this study are for use in the engineering evaluation of the embankments and reinforced-concrete structures.

This study consists of both a geological and a seismological analysis and includes the following: (a) a geological appraisal of the tectonics and the potentials for activity in the region, (b) a seismological appraisal of the historic seismicity, (c) an interpretation of seismic source areas and MCE with their prospects for recurrence, (d) attenuated peak ground motions at the site, and (e) accelerograms and response spectra for analogous cyclic shaking. The ground motions presented are in accordance with the requirements mandated by ER 1110-2-1806 of 31 July 1995.

Study Area

The study includes the geology, seismic tectonics, and earthquake potential within a radial distance of 150 km of the powerhouse.

Geology and Tectonics

The St. Stephen Powerhouse is in the Atlantic Coastal Plain about 60 km north of Charleston. Figure 1, from Klitgord, Dillon, and Popenoe (1983), shows schematically the geology of the region. The fall line separates the Coastal Plain from the ancient metamorphosed rocks of the Piedmont. There are two basement hinge zones. The hinge zone at the fall line is where the ancient

metamorphosed and crystalline rocks dip seaward and are overlain by the younger sedimentary deposits that comprise the Coastal Plain. Another hinge line at the edge of the Continental Shelf is where the dip steepens into the ocean and where the Coastal Plain is terminated.

The buried metamorphosed rocks beneath the sediments of the Coastal Plain show magnetic highs and magnetic lows. The ancient rocks contain the remnants of basins that resulted from late Triassic rifting. These show up as magnetic lows. Intrusive igneous rocks, which may be ancient, show up as magnetic highs. These heterogeneities beneath the blanket of Coastal Plain sediments may be responsible for concentrating stresses, the release of which causes fault displacements that extend into the overlying deposits and are locally the cause of earthquakes.

Shilt et al. (1983) ran reflection profiles through the Coastal Plain sedimentary layers. A probable boundary between lower Mesozoic sediments and crystalline basement or an older basalt is formed at about 1,400 m in the Charleston area. The profiles contain displacements that indicate faulting within the sedimentary section. The thickness of sedimentary rocks at the St. Stephen Powerhouse site is slightly less than that in the Charleston area, or about 1,000 m.

Foundation Lithology

The Powerhouse is in an area of the Coastal Plain where the surficial deposits are alluvial terraces and alluvium deposited in river valleys. Thicknesses of those deposits were determined by borings at the site and were found to be in the range of 80 to 100 ft. Preconstruction borings (Design Memorandum 6, 1975-1978) show a good correlation of materials from boring to boring throughout the site. Typically the section is composed of bedded sands and silts with interspersed clays and occasional lenses which contain crushed shells.

The bedrock sequence beneath the Powerhouse, as revealed by borings (Design Memorandum 6) is:

- a. Indurated clay shale, about 15- to 20-ft thick.
- b. A glauconite zone about 1/2-ft thick.
- c. Fossiliferous limestone or coquina, about 15-ft thick, (with the above glauconite zone, this is the bearing level for the Powerhouse).
- d. Thin sand layer, slightly calcareous and partially indurated about 5- to 10-ft thick.

- e. Limestone, about 25- to 30-ft thick. The limestone is a highly fossiliferous coquina through most of its thickness. The coquina is highly porous and is believed to be the main water-producing stratum in the section. The water is artesian.
- f. Sand, about 20-ft thick, slightly calcareous, and irregularly indurated. The lower 5 ft is shaley and grades into the underlying layer.
- g. Soft to medium hard, calcareous shale. The shale forms an aquiclude for the aquifer lying above. This shale is similar to the upper shale.

These sedimentary beds are generally flat-lying and are correlatable between borings. No fault displacements or other structural anomalies were observed in the borings and excavations made at the site.

Seismicity

Seismic History

A tabulation of earthquakes of Modified Mercalli (MM) intensity III and greater, recorded within 150 km of the St. Stephen Powerhouse, is shown in Appendix A. The data are from the National Geophysical Data Center of the National Oceanic and Atmospheric Agency in Boulder, CO. The years of coverage are from 1698 to 1993. Figure 2 shows the geographic distribution of these earthquakes. The location of the St. Stephen Powerhouse is indicated by a star. No earthquakes are shown within a radius of about 45 km from the Powerhouse. The principal source area of seismicity is a relatively small area of intense seismicity to the southwest of the Powerhouse. Another lesser and more diffused source lies to the west of the Powerhouse.

The earthquake information for this region prior to the 1960's was recorded as "intensity" which is a measure of how an earthquake is felt and the damage it does. The scale used is the MM of 1931, shown in an abbreviated form in Table 1. The scale is a subjective numerical index that ranges from I to XII. Intensity XII, or total destruction, is conceptual but almost never occurs.

Earthquake magnitudes are indirect measures of the energies released during earthquakes. The general relation between intensity and magnitude for a plate interior is shown in Table 2.

Earthquakes in this region can be inferred to result from one or more of the following possible causes.

a. Focusing of regional compressive stresses along the boundaries of heterogeneous rock masses and release of these stresses by movement through reactivation of ancient faults.

- b. Possible small-scale introduction of magma from great depth with an accompanying buildup of stresses.
- c. Focusing and release of regional stresses along ancient rifts which remain as zones of crustal weakness.
- d. Slow, very broad regional compression causing reactivation of ancient thrust faults.
- e. Extensional movement along a sagging graben with activation of normal faults.

There is no way that all of these theories can apply everywhere since the extensional and the compressional postulations contradict each other. Also, each of these theories can be interpreted as meaning that a major earthquake can happen at a location where no historic earthquake has occurred. That idea, though seemingly possible on the face of it, must be handled with care because it can mean that larger earthquakes will happen almost everywhere in this region and that is not what we observe elsewhere in the world. It is essential to concentrate on the experiences with earthquakes as the only direct clue to present-day tectonics. Earthquake- generating faults are not identifiable on the ground surface in the region. However, the areal distribution of earthquakes and their concentrations can be used to define locations and boundaries for seismic source zones.

Seismic Source Zones and Maximum Earthquakes

A seismic source zone is an inclusive area over which an earthquake of a given maximum size can occur anywhere. That earthquake is a floating earthquake. A seismic zone is supplemental to and can include faults that are the sources of earthquakes when they are identified. The purpose of such zones is to avoid surprises.

The seismic zones as constructed in this report represent present-day tectonism. These are zones that are not determined by tectonic and physiographic provinces or regional geologic structure since those are products of past tectonism.

Criteria for developing zonations are:

a. Zones that have great activity should be as small as possible. They are likely to be caused by a definite structure, such as a fault or a pluton, and activity should be limited to that structural association. Such a source may be a seismic hotspot. A seismic hotspot requires locally large historic earthquakes, frequent to continuous microearthquakes, and a well defined area. Maps of residual values for magnetometer and

Bouguer gravity surveys may provide structural information to corroborate the boundaries of hotspots.

- b. One earthquake can adjust a boundary to a seismic zone, but cannot create a zone.
- c. The maximum felt earthquake is equal to or less than the maximum zone earthquake.
- d. The maximum zone earthquake is a floating earthquake, one that can be moved anywhere in that zone.
- e. Assignment of the maximum zone earthquake is judgmental.

Figure 3 shows seismic zones with MM intensity values for maximum floating earthquakes. These are zones for potential earthquakes.

The severest seismic hazard is concentrated in two small seismic source zones at Summerville and Bowman. Summerville is given a peak intensity of IX to X based on the 1886 experience. Bowman has similar seismicity, but the earthquakes are much fewer and more restricted in area, thus a lower potential of VIII is assigned. The much broader and more encompassing zone that includes Columbia and Greenville includes widely scattered small earthquakes. The largest are M = 4.5-4.9. An earthquake of this size indicates a peak epicentral intensity of V to VI. The VI was raised to VII for conservatism. The adjacent zones are very nearly aseismic. However, as small earthquakes are known to occur even in the most aseismic areas, a base seismicity of VI is assigned. The VI is a level at which there is hardly any damage.

Figure 4, from Tarr and Rhea (1983), shows in greater detail the evidence for identifying and locating the heightened seismic potential at Summerville and Bowman. Note the interpretations for the seismicity at Summerville, Middleton Place, and Adams Run. The elongate ellipses represent interpretations of the fault zones along which the earthquakes are occurring. The interpretations are from fault-plane solutions made on microearthquakes, those that are $M \le 3.5$, recorded between March 1973 and December 1979. The exercise was to more accurately locate the source area for the Charleston earthquakes.

Tarr and Rhea (1983) believe that the observed activity in the Summerville-Middleton Place source area identifies the proper location of the Charleston earthquakes of 1886. They found a three-segment fault zone. The faults strike northwest and are steeply dipping at angles of 80° to 90°. The interpretation is that these are dip-slip motions. Events at Bowman and Adams Run are spatially distinct. No earthquakes were recorded in the gaps between these sources in 9 years of observations following 1971. The depths and intense clustering of the earthquakes indicate planes of weakness in crustal units of Mesozoic age.

The vertical sections in Figure 4 show that the earthquakes have focal depths to about 15 km and are broadly scattered in the vertical sections. These are earthquakes that reach far into the crystalline basement rocks where stress drops can be large enough to produce powerful earthquakes.

Appendix A shows that dozens of felt earthquakes occurred along with the Charleston event of 1886. There would have been thousands of microearthquakes shown had there been recordings made. Those earthquakes are still occurring, as shown in the work of Tarr and Rhea (1983). This meets the criteria for a seismic hotspot.

Appendix A lists four Charleston earthquakes of 1886 that range from VI to X. These are shown in Table 3 along with approximate coordinates for their epicenters.

In the isoseismal map shown in Figure 5, Bollinger (1977) reinterpreted the reports of ground shaking in 1886. His interpretation for the St. Stephen site is approximately an intensity VIII. This value can be corroborated by attenuating the intensity over the distance from the source to the site. A general distance, which can only be approximate for an intensity value, is given in Appendix A as 57 km. A rate of attenuation for the Eastern Province is given by Chandra (1979). Figure 6 shows this attenuation to be 1-1/2 intensity units. The intensity at the Powerhouse would be an MM 8.5.

Bollinger (1983) determined that the intensity data showed the 1 September 1886 Charleston earthquake to an $m_b = 6.7$. Table 2 shows this to be an M = 7.5. This magnitude value allows a determination to be made for magnitude-and-distance attenuations. These will be presented in this report under ground motions.

Table 3 shows that 11 historic earthquakes were felt at the St. Stephen Powerhouse site with intensities of IV and greater. Significantly, nine of these events came from the Summerville area. The 3 August 1959 earthquake came from a different source in the region. That earthquake was an intensity VI and it originated in the same intensity VI zone in which the Powerhouse is located. An intensity of MM VI was estimated by Stearns and Wilson (1972) for the effects in the area of the site of shaking from the major New Madrid event, around 800 km distant. The only serious shaking came from the 1 September 1886 Charleston earthquake and was an MM intensity of 8.5 at the site.

Earthquake Ground Motions

Maximum Credible Earthquake (MCE)

The MCE is the largest earthquake that can reasonably be expected. ER 1110-2-1806 (31 July 1995) mandates that for a critical structure MCE be

obtained by a deterministic analysis. The deterministic analysis is not timedependent, as is a probabilistic evaluation.

For the St. Stephen Powerhouse, MCEs would be as follows:

- a. An MM intensity X earthquake, M = 7.5, attenuated from the Summerville source for ~ 55 km to the site (see Figure 3).
- b. A floating earthquake of MM VI, M = 5.0.

Field Conditions

Ground motions from an earthquake source using MM intensity are characterized as being either near field or far field. Ground motions are different for each field type. Near field motions, those originating near the earthquake source, are characterized by a large dispersion in the peak ground motions which are caused by complicated reflection and refraction patterns, focusing effects of the waves, impedance mismatches, and resonance effects. In contrast, the wave patterns for far field motions are more orderly and they are more muted or dampened so that they are better predictable.

The limits of the near field are variable, depending on the severity of the earthquakes. The relationship between earthquake magnitude (M), epicentral intensity, and the limits of the near field are given in the following set of relations, see Krinitzsky (1995).

	Near Field	Limits
M	MM Maximum Intensity, Io	Distance from Source, km
5.0	VI	5
5.5	VII	15
6.0	VIII	25
6.5	IX	35
7.0	X	40
7.5	X-XI	45

Near field conditions are specified only when the site of interest is located within or near a seismic hotspot.

Though the Summerville source is a hotspot, its distance from the site requires the use of far field motions for the attenuated intensity level. A mean plus standard deviation (SD) is used to encompass the range of strong motion values and to provide a practical level for engineering.

For the floating earthquake of MM VI in the zone of the St. Stephen Powerhouse, a far field set of motions would be used. The principle is that the earthquake, even if it were to happen at the site, would be at a focal depth at or greater than the near field limit for a MM VI.

Table 4 gives parameters for peak MCE ground motions in the free field at the St. Stephen Powerhouse site. The parameters are for selecting and adjusting strong motion records to use in engineering analyses. The ground motions were obtained from intensity-based charts (Krinitzsky 1995) and magnitude-distance charts (Krinitzsky 1995). The intensity charts are shown for acceleration, velocity, and duration for hard sites in Figures 7 to 9 and for soft sites in Figures 10 to 12. Ground motion charts for magnitude and distance for hard sites are shown in Figures 13 to 15. Soft sites are shown in Figures 16 to 18.

A site is soft when it has a surface layer ≥ 16 m, in which the shear wave velocities are less than 400 m/sec. A hard site is where the shear wave velocities are greater than 400 m/sec and overlying soft layers with smaller shear wave velocities are less than or equal to 15 m in thickness.

The earthquakes in South Carolina are interpreted to be shallow crustal events for which the focal depths are ≤ 19 km.

MCE Analogous Time Histories

The charts in Figures 7-18 show peak values and catalogue numbers for selected strong motion records. The catalogue is by Leeds (1992) and is a collection of recommended accelerograms and response spectra. Figures 19 to 30 show a selection of records that can be used. They are:

- Figure 19. San Fernando Earthquake, 9 February 1971, 535 S. Fermont AV., Basement, CAL 61.
- Figure 20. Superstition Mountain, 15 October 1979, CAL 139.
- Figure 21. Coalinga, 2 May 1983, Parkfield Fault Zone 14, 90 Deg CAL 189.
- Figure 22. Coalinga, 2 May 2 1983, Parkfield Fault Zone 14, 0 Deg, CAL 190.
- Figure 23. Santa Cruz Mountains, Loma Prieta, 17 October 1989, San Francisco International Airport, CAL 391.
- Figure 24. Morgan Hill Earthquake, 24 April 1984, Gilroy No. 7, CAL 216.
- Figure 25. Morgan Hill Earthquake 24 April 1984, Coyote Lake Dam, CHN-1, 285 Deg, CAL 228.

- Figure 26. Morgan Hill Earthquake, 24 April 1984, Coyote Lake Dam, CHN-3, 195 Deg, CAL 229.
- Figure 27. Whittier Earthquake, 1 October 1987, Tarzana, Cedar Hill Nursery, CHN 1, 90 Deg, CAL 270.
- Figure 28. Whittier Earthquake, 1 October 1987, Tarzana, Cedar Hill Nursery, CHN-3, 0 Deg, CAL 271.
- Figure 29. Sturno, Italy, 23 November 1980, N-S Component, ITA 20.
- Figure 30. Sturno, Italy, 23 November 1980, E-W Component, ITA 21.

All of the records are for hard sites except those in Figures 19 and 20, which are for soft sites. Additional hard site records were extracted from the USGS database of strong ground motion recordings to find records that best fit the target ratio of peak ground acceleration (PGA) and peak ground velocity (PGV), magnitude, distance, and response spectra (described in the next section). The records considered are listed in Table 5. Among these records, three appeared to be particularly promising because of the PGA to PGV ratio:

- 1. Loma Prieta Gilroy #7, 0 degree component.
- 2. Coalinga Earthquake, Parkfield Fault Zone 14.
- 3. San Fernando Earthquake, 234 Figuero.

The acceleration histories and Arias intensities for these records are shown in Figures 31, 32, and 33, respectively. Duration of strong motion is shown in two forms on these figures:

- 1. The duration of motion exceeding 0.05 g.
- 2. The duration of Arias intensity from 5 to 95% of total energy

By both duration definitions, the three records have durations ranging from 11 sec to 18 sec, which is reasonably consistent with the target duration for a hard site. The Loma Prieta and Coalinga records are from M = 6.5 events, somewhat less than the target MCE magnitude of 7.5. This is reflected in the total Arias intensity delivered during the period of strong motion and total energy. The Loma Prieta Gilroy # 7 record has total Arias intensity of 101 cm/sec, with 91 cm/sec delivered in the duration of strong motion (defined as 5 to 95% of total energy delivered) of 11.5 sec. The Coalinga Fault Zone 14 record has 67 cm/sec total Arias intensity with 56 cm/sec delivered in a duration of 13.38 sec. The San Fernando Gilroy # 7 record has 73 cm/sec total Arias intensity with 65 delivered in a duration of 11.3 sec.

MCE Response Spectra

The response spectra for the MCE was estimated from spectral attenuations developed for Eastern North America, specifically Atkinson and Boore (1995) and Toro, Abrahamson, and Schneider (1997), using the sources zones described earlier. The Toro-Abrahamson spectra generally exceed the Atkinson and Boore spectra at periods exceeding 0.1 sec, and are recommended for the MCE response spectra. Figure 34 shows the Toro-Abrahamson spectra for damping ratios of 2, 5, 10, and 15%. Figures 35, 36, and 37 show the mean and meanplus-sigma response spectra from these two relationships for a damping of 5%, with the response spectra of the three acceleration histories superimposed, Loma Prieta record in Figure 35, Coalinga record in Figure 36, and San Fernando record in Figure 37. All three records have high energy content in the period range of about 0.1 sec to 2 sec, and generally trace the target mean-plus-sigma response spectra.

Operating Basis Earthquake (OBE)

The OBE is an earthquake that allows damage, providing there is no hazard to human life, and permits the structure to remain operational with repairs. Further, it is an earthquake that is expected to occur during the life of the structure. According to ER 111-2-1806, the OBE may be determined either deterministically or probabilistically. The actual values of the OBE motions are based on economic considerations, but typically they correspond to ground motions with a return period of exceedance of about 144 years. For this study, the OBE ground motions were selected from the USGS maps (dated November 1996) available over the Internet. These maps provide detailed probabilistic seismic hazard information on the resolution of 0.1 degree latitude by 0.1 degree longitude for return periods of 475, 975, and 2,475 years. The USGS maps provide peak ground acceleration (PGA) for various site conditions as well as equal hazard spectral ordinates (SA) for periods of 0.2, 0.3, and 1.0 sec. Earthquake Design Guidance for Structures (EDGS), Developing Standard Response Spectra and Effective Peak Ground Accelerations for Use in the Design of Civil Works Projects, dated October 1996, recommends extrapolating the data on a log-log plot to estimate spectral ordinates and PGA for other return periods. The resulting seismic hazard curves are shown in Figure 38 and listed in Table 6. Since the points are generally not colinear on a log-log plot, extrapolation using all three return periods is slightly different from using only the nearest two data points. These two extrapolations are shown in Figure 38, and result in the range of values listed in Table 6.

USGS-National Seismic Hazards Mapping Project-Deaggregated Seismic Hazard

Extracted from National Hazard Mapping Project, USGS www home page:

At 56 cities in the Central and Eastern U.S. (CEUS) and 44 cities in the Western U.S. (WUS), the seismic hazard corresponding to a 2% probability of exceedance in 50 years is deaggregated by magnitude (Mw, or moment magnitude) and by epicentral distance (CEUS) or hypocentral distance (WUS). Hazard with respect to magnitude is binned into intervals of width 0.5 Mw. Hazard with respect to epicentral distance is binned into intervals of 25 km width. The hazard probabilities are deaggregated for the following ground motion parameters: PGA, 1.0, 0.3, and 0.2 second PSA (Note: This corresponds to PGA in text.).

Four matrices of percent contribution to hazard are available at this web site. The matrices are organized with magnitude intervals corresponding to columns and distance intervals corresponding to rows. The first row of numbers gives the upper endpoint of the magnitude interval. For example, the number 6 means that seismic sources with magnitudes in the interval 5.5 < Mw <= 6.0 are included in hazard calculations for that column. The first column of numbers gives the upper endpoint of the epicentral distance interval. For example, the number 150 means that source-to-station distances in the interval 125 < d <= 150 km are included in the hazard calculations for that row. Missing rows, or gaps in the matrix, correspond to distance ranges for which the greatest percent contribution to hazard is less than 0.0005, yielding a row of zeros to the level of precision given in the below data.

For the CEUS, the lowest magnitude considered for hazard calculations is MbLg 5.0. This magnitude corresponds to Mw = 4.7 using the Johnston (1996) relationship between the two magnitudes. Thus, for CEUS cities, the interval width for the first column of contribution to hazard is about 0.3 Mw units, rather than 0.5 units, the usual interval width. For the WUS, the lowest magnitude considered for hazard calculations is Mw = 5.0. The entries are percent contribution to hazard. They will sum to 100 percent for each matrix.

The deaggregated matrices for Charleston, SC, are provided in Table 7 for PGA and SA at 1, 3.33, and 5 Hz (periods of 1, 0.3, and 0.2 sec), for a return period of 2,475 years. Examination of the table indicates that the majority of seismic hazard comes from nearby zones, within 25 to 50 km, as expected from the seismic history, and as identified earlier in this chapter. The deaggregated

matrices are plotted in Figure 39 for PGA and Figures 40-42 for the spectral ordinates.

Previous Interpretation of Ground Motions

Previous interpretations of ground motion parameters for use at the Cooper River Rediversion Project, of which the St. Stephen Powerhouse is a part, are as follows:

- a. In a letter of 22 December 1981 to Mr. Harry E. Thomas, FERC,
 Washington, DC, from Otto W. Nuttli, H. Bolton Seed, and Stanley D.
 Wilson, the following reasonings were presented:
 - (1) A Charleston, SC, earthquake was postulated at a distance of 65 km. MM intensities of IX to X should be constant to 25 km and fall off to IX at 45 km.
 - (2) The design motions should be for a Charleston earthquake, M = 7, 15 mi from the Pinopolis West Dam. (The Pinopolis West Dam is about 10 km from the St. Stephen Powerhouse.)
 - (3) Peak acceleration at the site is 0.30 to 0.35 g.
- b. In a meeting with FERC on 2 September 1982 in Washington, DC, re the Santee North and Pinopolis West Dams, the following values were recommended:
 - (1) A magnitude at the source of 7.5.
 - (2) Acceleration = 0.45g. Motion for a rock outcrop near the dam. Duration = 25 sec (≥ 0.05g).
- c. In a report of 10 June 1986 to Mr. Ronald A. Corso, FERC, Washington, DC, from Dr. A. J. Hendron, Jr., the following recommendation was made for the Pinopolis West Dam:
 - (1) Acceleration = 0.45 g, Velocity = 22 in./sec

The reasoning for the values was that 1 g has a velocity of 48 in./sec; proportional scaling provided the above parameters.

(2) Use the Taft and Castaic records. Both records have single high peaks of 0.45 g.

3 Newmark-Sliding-Block Type Deformation Analysis of Embankments

Background

A Newmark-sliding-block type of deformation analysis models the displacing part of an embankment as a rigid block sliding on an inclined plane (Newmark 1965). This type of analysis is appropriate for an embankment dam if the embankment and its foundation soils are not expected to suffer liquefaction or severe softening under cyclic loading due to earthquake shaking, as is the case assumed at the St. Stephen Powerhouse Project. Other contributions to a coherent procedure using the sliding block approach have been made by Taylor and Whitman (1952), Ambraseys and Sarma (1967), Sarma (1975, 1979), Goodman and Seed (1966), Makdisi and Seed (1977), Franklin and Chang (1977), Franklin and Hynes-Griffin (1981), and Hynes-Griffin and Franklin (1984).

Shearing resistance between the potential sliding mass and the underlying base is evaluated in terms of a yield acceleration, k_y , defined as the acceleration of the sliding mass that will reduce the factor of safety against sliding to unity, i.e., that will make sliding imminent. The value of k_y is expressed as a fraction of gravity (g) and is obtained through a traditional limit equilibrium slope stability analysis that applies the seismic load horizontally at the center of gravity of the sliding mass. Spencer's method (1967) in the computer program UTEXAS3TM (developed by Stephen G. Wright at the University of Texas at Austin), adapted for microcomputer use as documented by Edris and Wright (1992), was used in this study.

An analysis of the amplification response of the embankment is typically incorporated to account for amplified accelerations in the embankment. Amplifications were estimated from empirical observations of dynamic response of embankments (Harder 1991), SHAKE analyses, and charts developed by Makdisi and Seed (1977) from numerous finite element response analyses of embankment dams founded on rock.

Because the amplified accelerations vary over the height of the embankment, yield accelerations were determined for possible sliding masses whose bases lie at various elevations in the idealized sections, both upstream and downstream.

Displacement charts have been developed for Newmark-sliding-block models by Makdisi and Seed (1977) and Hynes-Griffin and Franklin (1984). The Makdisi and Seed displacement charts were used in this study since they include the effect of earthquake magnitude and frequency changes due to amplification in the embankment.

Sections Selected for Analysis

A plan of the project is shown in Figure 43. Three sections were considered in the deformation analysis: Section 1, estimated to be the most vulnerable embankment section founded on natural soil deposits; Section 2, estimated to be the most vulnerable upstream section through a retaining wall; and Section 3, the maximum section of the embankment dams flanking the Powerhouse structure. The locations of these sections are shown in Figure 43. Section 1, as idealized, is shown in Figure 44. Section 2, as idealized, is shown in Figure 45. Section 3, as idealized, is shown in Figure 46. The material properties for the zones shown in Figures 44-46 were derived from the existing project documentation and are listed in Table 8.

Yield Accelerations

Yield accelerations were computed with Spencer's method in UTEXAS3. The slip surfaces with minimum yield accelerations at a given elevation are shown in Figure 47 for Section 1, Figure 48 for Section 2, and Figure 49 for Section 3. The computed yield accelerations for these sections are shown in Figures 50-52. Also shown are the computed static factors of safety.

Dynamic Response

Makdisi and Seed (1977) developed charts for dynamic response of embankment dams founded on rock from numerous finite element response analyses. In these analyses, the earthquake-induced acceleration applied to the sliding mass is interpreted by summing the contributions from the elements along the potential sliding surface, as proposed by Chopra (1966). Figure 53 shows the Makdisi-Seed dynamic response chart, which gives the summed acceleration applied to the sliding surface, k_{max} , divided by the peak crest acceleration, u_{crest} , expressed for surfaces at different depths in the embankment, as a ratio of depth of sliding surface, y, to embankment height, h.

Use of the Makdisi-Seed response chart requires estimation of the crest acceleration. Harder (1991) collected empirical observations of crest to base or abutment accelerations and developed the upper-bound chart shown in Figure 54. The data from the U.S. Army Engineer Corps Strong Motion Instrument Program (SMIP) database for seismic response of Corps dams, current through 1996, have been added to this figure. For a base acceleration of about 0.33g as recommended in Chapter 2, the corresponding upper-bound crest acceleration is 0.64g.

The Makdisi-Seed chart was derived for embankments founded on rock. For embankments founded on soil deposits, it requires some estimation of appropriate effective embankment height and crest acceleration to use in estimating k_{max} . SHAKE analyses were also performed to estimate k_{max} and u_{crest} , using the Corps program WESHAKE. Although WESHAKE is a onedimensional wave propagation code, it provides a fairly good approximation of the dynamic response at depth (error is typically greatest in the top 10 to 20% of height of the column (Elton, Shie, and Hadj-Hamou 1991), particularly for slip surfaces passing through natural materials. The WESHAKE results in this study were also used to estimate k_{max}. The WESHAKE columns and estimated shear wave velocity profiles are shown in Figure 55. Shear wave velocities were estimated from the WES shear wave velocity data base and Cone Penetrometer Test (CPT) data base using Standard Penetration Test (SPT) blowcounts reported in the project documentation. The accelerogram used in the computations was the Loma Prieta Gilroy # 7 record described in Chapter 2. The WESHAKE results, acceleration, cyclic shear stress, and cyclic shear strain plotted versus depth, are shown in Figures 56-59. The k_{max} values, estimated from both the Makdisi-Seed chart and the WESHAKE results, are shown in Figures 60-63.

Section 1, embankment on natural ground. The crest acceleration for the dike was estimated from the free-field WESHAKE analysis which indicates a base acceleration of about 0.5g. The corresponding crest acceleration is about 0.7g from Figure 54. This results in the k_{max} values shown in Figure 60, estimated from the Makdisi-Seed chart in Figure 53. The WESHAKE analysis of this section assumed a possible zone of low velocity in the natural materials. If such a zone exists, it is unlikely that such high levels of acceleration could be transmitted to the dike. Consequently, the displacements were calculated using k_{max} values from both the Makdisi-Seed approach as well as the WESHAKE values plotted in Figure 60.

Section 2, upstream retaining wall. The crest acceleration for the retaining wall section was estimated as 0.64g from Figure 54, with a base acceleration of 0.33g, observed in the WESHAKE analysis. The k_{max} values from both the Makdisi-Seed approach and the WESHAKE calculations are plotted in Figure 61. Since all the yield surfaces passed below the wall, below the effective height, a constant value of k_{max} at a depth of 72 ft from the Makdisi-Seed approach was used in the displacement calculations.

Section 3, maximum embankment section flanking Powerhouse, upstream surfaces. The WESHAKE and Makdisi-Seed estimates for k_{max} values are plotted in Figure 62. Two effective heights were used in estimating the Makdisi-Seed values, 115 ft corresponding to the height of the crest above the shale bedrock base, and 64 ft, an average height of embankment above intake and tailrace elevations.

Section 3, maximum embankment section flanking Powerhouse, downstream switchyard surfaces. The WESHAKE and Makdisi-Seed estimates for k_{max} values are plotted in Figure 63. Because the switchyard is a fairly large, level ground area, the ground surface acceleration from WESHAKE was used to estimate the crest acceleration for the Makdisi-Seed k_{max} values, hence the close agreement between both approaches to estimate k_{max} .

Deformation Estimates

The Makdisi-Seed deformation chart, shown in Figure 64, was developed specifically for embankment dams founded on rock, as is the case for the main flanking embankments at the St. Stephen Powerhouse Project. The Hynes² Franklin displacement chart (after Hynes-Griffin and Franklin 1984) is shown in Figure 65 for comparison. The upper-bound displacement curve in the Hynes-Franklin chart generally corresponds to magnitude 7.5 earthquakes, and falls slightly below the average of the magnitude 7.5 relationship in the Makdisi-Seed chart. This difference is due in part to the integration scheme used to develop the chart, as well as the fact that the Makdisi-Seed chart uses response accelerograms computed in FLUSH throughout the embankment, whereas the Hynes-Franklin chart is computed directly from the recorded accelerogram. Since the difference is greatest at small levels of displacement, the Makdisi-Seed chart was used in the displacement computations. The displacement results are plotted in Figures 66-68.

Section 1, embankment on natural ground. Deformations and yield surfaces for this section are plotted in Figure 66. The yield surfaces for the dike section all pass beneath the embankment through natural soil deposits. The properties of these materials were estimated from other locations at the site since no direct measurements were available in the documentation. With these estimated strengths, the largest deformation is estimated to be about 16 to 34 cm. Better information about the natural soils may significantly reduce these deformation estimates.

<u>Section 2, upstream retaining wall</u>. Deformations and yield surfaces for this section are plotted in Figure 67. The maximum displacement estimated was 20 cm for surfaces passing through select fill beneath the retaining wall.

Section 3, maximum embankment section flanking Powerhouse, upstream surfaces. The displacements for this section are plotted in Figure 68. For an effective height of 64 ft, the displacements are zero, since the yield accelerations

exceed the estimated k_{max} values. For an effective height of 115 ft, which should be conservative, the maximum displacement is less than 1 cm.

Section 3, maximum embankment section flanking Powerhouse, downstream switchyard surfaces. The displacements for this section are plotted in Figure 68. The yield acceleration for these surfaces all exceeded estimated k_{max} values. Consequently, displacements for this section are zero.

Damage Assessment

For Section 3, the maximum section for the embankments flanking the powerhouse, zero to negligible (less than 1 cm) permanent displacements are expected for the assumed material properties and input motions, using maximum crest accelerations from empirical response charts. For the other sections, the dike and the retaining wall, deformations on the order of 15 to 30 cm were calculated, again using fairly conservative estimates of response. Deformation levels on this order are generally assumed to be acceptable, with no threat to reservoir retention.

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Table 1 Abbreviated Modified Mercalli 1931 Intensity Scale

- I. Not felt except by a very few under especially favorable conditions.
- II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
- III. Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing automobiles may rock slightly. Vibration like passing of truck. Duration can be estimated.
- IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing automobiles rocked noticeably.
- V. Felt by nearly everyone; many awakened. Some dishes, windows, and other fragile items broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles and other tall objects sometimes noticed. Pendulum clocks may stop.
- VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
- VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-build ordinary structure; considerable in poorly built or badly designed structures. Some chimneys broken. Noticed by persons driving automobiles.
- VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse. Great damage in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving automobiles disturbed.
- IX. Damage considerable in specially designed structures; well-designed frame structures thrown out-of-plumb; damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

(Continued)

Table 1 (Concluded)

- X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed. Ground badly cracked. Railroad rails bent. Many landslides on river banks and steep slopes. Shifted sand and mud. Water splashed over banks of rivers and lakes.
- XI. Few structures remain standing. Unreinforced masonry structures are nearly totally destroyed. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Railroad rails bent greatly.
- XII. Damage total. Waves apparently seen on ground surfaces. Lines of sight and level appear visually distorted. Objects thrown upward into the air.

Table 2
Equivalences Between Magnitude Scales and Intensity (Magnitudes were Modified from Nuttli and Shieh (1987). From Krinitzsky (1995)

				Plate Inte	rior	
M	m_{b}	M _L *	M _s	M _w	M ₀ (dyne-cm)	Epicentral Intensity MM
4.3	4.0		2.9	3.8	1021	IV
4.8	4.5		3.4	4.1	10 ²²	V
5.1	5.0		4.4	4.8	10 ²³	VI
5.4	5.5		5.4	5.4	10 ²⁴	VII
6.4	6.0		6.4	6.1	10 ²⁵	VIII
7.4	6.5		7.4	6.8	10 ²⁶	IX-X
8.4	7.0		8.4	7.4	10 ²⁷	XI-XII
* M _L g	generally r	not used in	plate in	terior.		

Table 3 Modified Mercalli, $I_s \ge IV$ at the St. Stephen Powerhouse Site. Data from IGDA/NOAA and Visvanathan (1980)

Date of Earthquake	Coordina	ites	I_o	Distance from Site km	Is
Dec 16, 1811	New Madrid, MO		XI-XII	800	IV*
Sep 1, 1886	32.9 N	80. W	X	57	VIII**
Sep 21, 1886	32.9	80	VI	57	IV
Oct 22, 1886	32.9	80	VII	57	V
Nov 5, 1886	32.9	80	VI	57	ΙV
June 12, 1912	32.9	80	VII***	57	V
Aug 3, 1959	33.	79.5	VI	61	IV
Mar 12, 1960	33.07	80.12	v	42	IV
Feb 3, 1972	33.31	80.58	v	44	IV
Nov 22, 1974	32.9	80.14	VI	60	IV
Sep 21, 1992	32.05	80.11	v	44	IV

^{*} Stearns and Wilson (1972). ** Bollinger (1977). ** Visvanathan (1980).

Table 4 Free Field Egk Ground Motio Rediversion Project	ns for MCE at St.	Stephen Powerl	nouse, Cooper River
	Accel, cm/sec ²	Vel, cm/sec	Dur ≥ 0.05g, sec
$I_o = X (10)$, Far Field, mean + S.D., Distance = 55 km, Chandra Intensity Attenuation $I_s = (8.5)$	n = 1.5 units.		
Soft Site	330	48	23
Hard Site	340	30	24
Magnitude = 7.5*, Attenuated 55 km			
Soft Site	330	52	60
Hard Site	320	23	18
* Bollinger (1983) pg T1: M _b	= 6.7, equivalent	to $M = 7.5$.	

Table 5 Step	Stephen Pt Powerh	owerho	use Ear	ouse Earthquake Time History Selection - Hard Sites	History S	election -]	Hard Si	es	
Earthquake Station Comp	EPI dist,km	Mag	Int	Amax, cm/s² (Scale Factor)	Vmax, cm	ΑΛ	Site	Selection Basis	File Shake Fok#
Target	50	7.5		330	48	11	hard	#Mag	
	50		Is8.5	320	23	14	hard	#Int	
Recorded Strong Motion Time Histories	ion Time H	istories							
San Fernando 234 Figuero	41	6.5ML	[o11	195.6 (1.67)	16.8	11.6	H,b-1 S4	Dbase	USACA02.055 Cai58 DB#1
Imperial Val Superstition Mtn	58	6.6ML	1009	189.2 (1.69)	9.0	21.0	H,f+1 S1	Mag	USACA24.058 Cal139
Loma Prieta Golden Gate	100	7.1ML	1008	238.8 (1.37)	35.5	6.7	H,brdg S1	Dbase	USACA57.072 Cal349
Coalinga Fault Zone 14	41	6.5		268.4 (1.20)	28.8	9.3	Н	Mag	USACA52.124 Cal189 DB#3
(43)	6633	6		257.0 (1.26)	35.4	7.3	Н	Mag	USACA52.125 Ca1190
Campania-Luciana Sturno NS	35	6.5ML	1009 1s08	220.8* (1.47)	42.2*	5.2	Н	ĮĮ.	ITA03.006 ITA20
"" WE	(0)	.	6	327.6* (0.99)	70.2*	4.7	69		ITA03.006 ITA21

NOTES: { +Dbase query for (epi:20-70) & (H) & (a/v:10-14)} {*uncorrected} {# KCN Charts}

Table 5 Stepl	Stephen Pt Powerk		ise Eari	ouse Earthquake Time History Selection - Hard Sites	History Se	lection - I	Iard Sit	es	
Earthquake Station Comp	EPI dist,km	Mag	Int	Amax, cm/s² (Scale Factor)	Vmax, cm	Α/V	Site	Selection Basis	File Shake Eqk#
Loma Prieta Gilroy#7 ODeg	24	7.1Ms	Io08 Is07	205.6 (1.56)	16.6	12.4	Н	+Dbase	GILROY#7.v2 Cal381 DB#2
 90Deg	(43)	(0)	(63)	314.3 (1.03)	16.3	19.3	6633	(63)	GILROY#7.v2 Cal381
Loma Prieta SFO TransAm bld	61	7.1	lo8 Is6	104 (3.12)	8.8	11.8	H,bldg	+Dbase	USACA57.060 Cal344
Morgan Hill Coyote Lake Dam	25	6.2ML	Io07	639.8 (0.51)	51.9	12.3	H,abut S2	+Dbase	USACA36.005 Cal229
Whittier Narrows Cedar Hill Nur. 90	43	5.9ML	1008	526.9 (0.62)	24.2	21.8	H S4	Int	USACA39.013 Ca1270
0 ""	(63)	69		397.5 (0.82)	19.2	20.7	(0)	(43)	USACA39.013 Cal271

NOTES: { +Dbase query for (epi:20-70) & (H) & (a/v:10-14)} {*uncorrected} {# KCN Charts}

	owerhouse, SC Hazard Spectra	•	+33.4 Longitu	ıde: - 79.9 vember 1996 M	aps	
Return Period	Annual	Peak Ground Acceleration	Peak Spectral Acceleration (g's)			
(yr)	Frequency of Exceedence	(g's)	0.2 sec	0.3 sec	1.0 sec	
475	0.0021 0.16		0.305	0.230	0.070	
975	0.0010	0.36	0.680	0.530	0.190	
2475	0.0004	0.84	1.590	1.240	0.460	
Extrapolated						
144	0.0069	0.041-0.050	0.013-0.019	0.056-0.068	0.081-0.095	

					arleston, SC Period 2475	
Distance			Moment	Magnitude		
(km)	5	5.5	6	6.5	7	7.5
25	4.646	0	5.669	5.162	3.491	56.908
50	0	0.123	0,462	1.088	1.616	15.414
75	0	0.002	0.015	0.086	0.266	3.681
100	0	0	0.001	0.011	0.055	0.816
125	0	0	0	0.003	0.019	0.364
150	0	0	0	0.001	0.006	0.082
175	0	0	0	0	0.002	0.007
200	0	0	0	0	0	0.002
225	0	0	0	0	0	0.001

T % (able 7b. D Contribution	eaggregat 1 to Hazard	ed Seismi to SA of 1	c Hazard C Hz for Retu	harleston, rn Period 2	SC 475 yrs
Distance			Moment	Magnitude		
(km)	5	5.5	6	6.5	7	7.5
25	0	0.032	0.691	2.538	2.994	56.841
50	0	0.001	0.064	0.697	1.837	19.594
75	0	0	0.006	0.126	0.579	7.96
100	0	0	0.001	0.033	0.209	2.831
125	0	0	0	0.016	0.114	1.836
150	0	0	0	0.008	0.063	0.608
175	0	0	0	0.004	0.036	0.085
200	0	0	0	0.002	0.023	0.028
225	0	0	0	0.001	0.017	0.022
250	0	0	0	0.001	0.011	0.016
275	0	0	0	0	0.006	0.013
300	0	0	0	0	0	0.013
325	0	0	0	0	0	0.01
350	0	0	0	0	0	0.007
375	0	0	0	0	0	0.006
400	0	0	.0	.0	0	0.005
425	0	0	0	0	0	0.004
450	0	0	0	0	0	0.004
475	0	0	0	0	0	0.003
500	00	0	0	0	0	0,003

.

	able 7c. Dontribution					
Distance			Moment	Magnitude		
(km)	5	5.5	6	6.5	7	7.5
25	0	1.074	2.937	4.099	3.378	57.495
50	0	0.034	0.281	1.042	1.902	17.925
75	0	0.001	0.017	0.136	0.464	5.85
100	0	0	0.002	0.026	0.133	1.727
125	0	0	0.001	0.01	0.062	1.002
150	0	0	0	0.004	0.028	0.29
175	0	0	0	0.001	0.012	0.032
200	0	0	0	0.001	0.006	0.008
225	0	0	0	0	0.003	0.005
250	0	0	0	0	0	0.005
275	0	0	. 0	0	0	0.003
300	0	0	0	0	0	0.001
325	0	0	0	0	0	0.001

Ta % C	able 7d. D	eaggregat 1 to Hazard	ed Seismid to SA of 5	c Hazard C Hz for Retu	Charleston, rn Period 24	SC 175 yrs
Distance			Moment	Magnitude		
(km)	5	5.5	6	6.5	7	7.5
25	2.18	0	3.931	4.413	3.347	57.418
50	0	0.076	0.388	1.097	1.784	17.042
75	0	0.002	0.023	0.13	0.395	5.165
100	0	0	0.002	0.022	0.104	1.417
125	0	0	0.001	0.007	0.045	0.749
150	0	0	0	0.002	0.018	0.199
175	0	0	0	0.001	0.007	0.02
200	0	0	0	0	0.003	0.005
225	0	0	0	0	0	0.004
250	0	0	0	0	0	0.002
275	0	0	0	0	0	0.001

Table 8 - Static Soil properties

	Material type	Layer to layer elevation	total unit weight	Drained soil properties	Soil strengths used for slope stability calculations
		Interface (feet)			Undrained soil properties
Select and pervious fill	ervious fill		120 pcf	$\varphi_d=35^\circ$	$\phi_{\rm u} = 35$
Impervious fill	611		120 pcf	$\Phi_d = 28^{\circ}$	$\phi_{\rm u} = 13^{\rm o} c_{\rm u} = 600 \rm psf$
Zone II fill			120 pcf	$\varphi_d=32^\circ$	$\phi_{\rm u} = 23^{\circ} c_{\rm u} = 400 \rm psf$
Zone I fill		402	125 pcf	$\varphi_d = 31^{\circ}$	$\phi_{\rm u} = 13^{\circ} c_{\rm u} = 600 \rm psf$
Upper natural soil zone	al soil zone	/U.II.	120 pcf	$\varphi_d = 28^{\circ}$	$\phi_{\rm u} = 24^{\circ} c_{\rm u} = 700 \rm psf$
Middle	Non horizontal layers	1	110 pcf	$\phi_d = 26^\circ$	$\phi_{\rm u} = 13^{\rm o} c_{\rm u} = 500 \rm psf$
natural soil zone	Short horizontal layers	18 ft	110 pcf	$\Phi_d = 18^{\circ}$	$\phi_{\rm u} = 13^{\circ} c_{\rm u} = 500 \text{psf}$
Lower natural soil zone	al soil zone	000	115 pcf	$\phi_d = 28^\circ$	$\phi_{\rm u} = 15^{\rm o} c_{\rm u} = 800 \rm psf$
Shale	•	-20 II	105 pcf	$\Phi_{\rm d} = 28^{\rm o} c_{\rm d} = 1000$	$\phi_{\rm u} = 20^{\circ} c_{\rm u} = 2600 \rm psf$
Limestone		-41 11	135 pcf	$\phi_d = 28^{\circ} c_d = 5700$	$\Phi_{\rm u} = 37^{\rm o} c_{\rm u} = 5700 \rm psf$

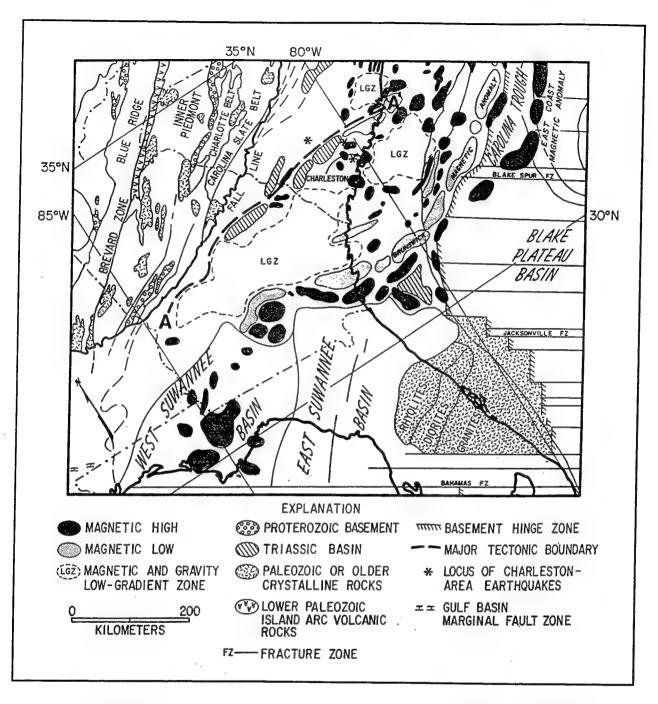
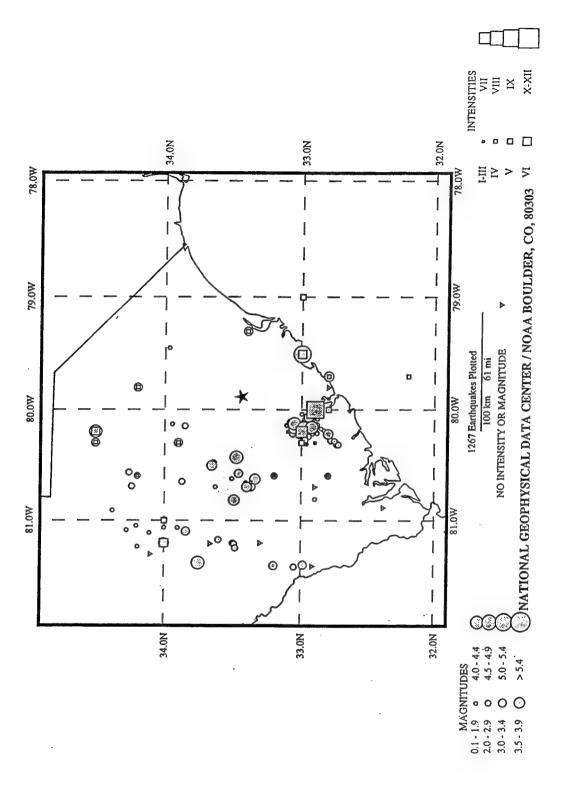


Figure 1. Geology and tectonism in the Charleston, South Carolina, region. From Klitgard et al. (1983).



Historic seismicity within 150 km of the St. Stephen Powerhouse (shown with a star). The data are listed in Appendix A. Figure 2.

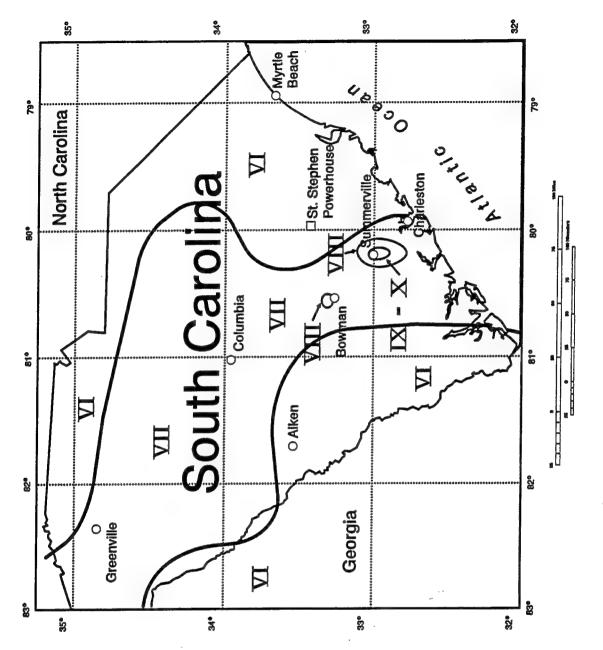


Figure 3. Seismic source zones in South Carolina.

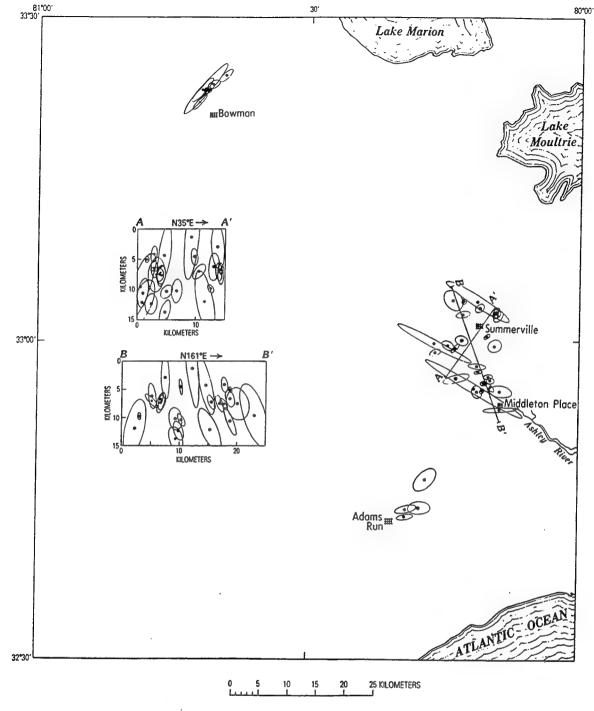


Figure 4. Locations of earthquakes and their hypocenters near Charleston, South Carolina. The data are from recordings made between March 1973 and December 1979. From Tarr and Rhea (1983).

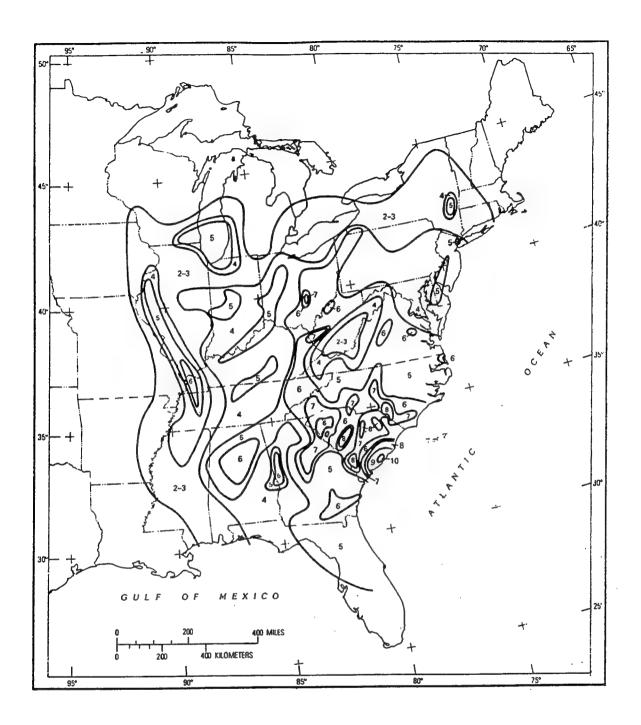


Figure 5. Distribution of Modified Mercalli intensities for the Charleston, South Carolina, earthquake of September 1, 1886. From Bollinger (1977).

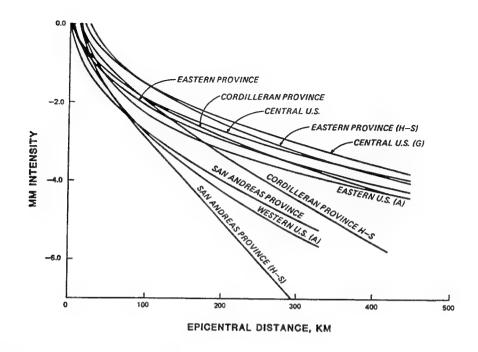


Figure 6. Attenuation of MM intensities with distance in various areas of the United States. From Chandra (1979).

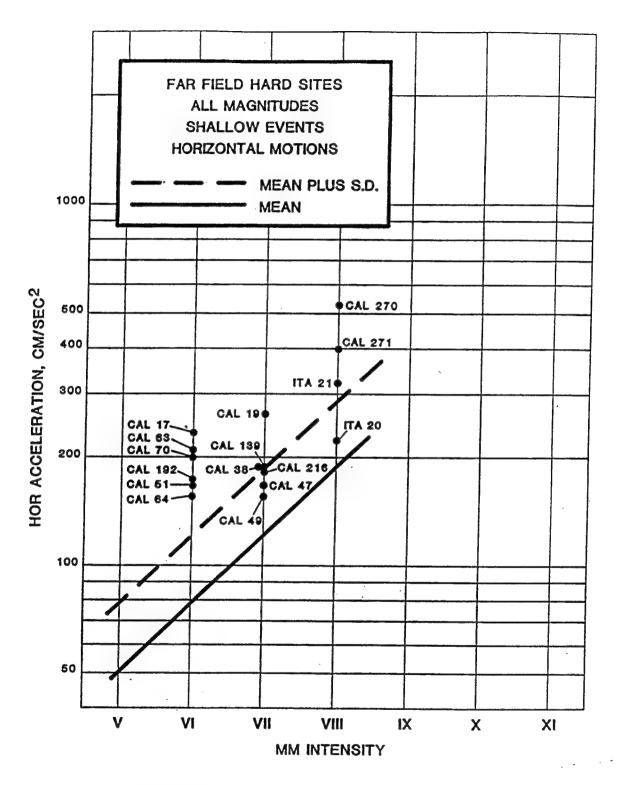


Figure 7. Accelerograms for acceleration and intensity for shallow earthquakes at far-field hard sites.

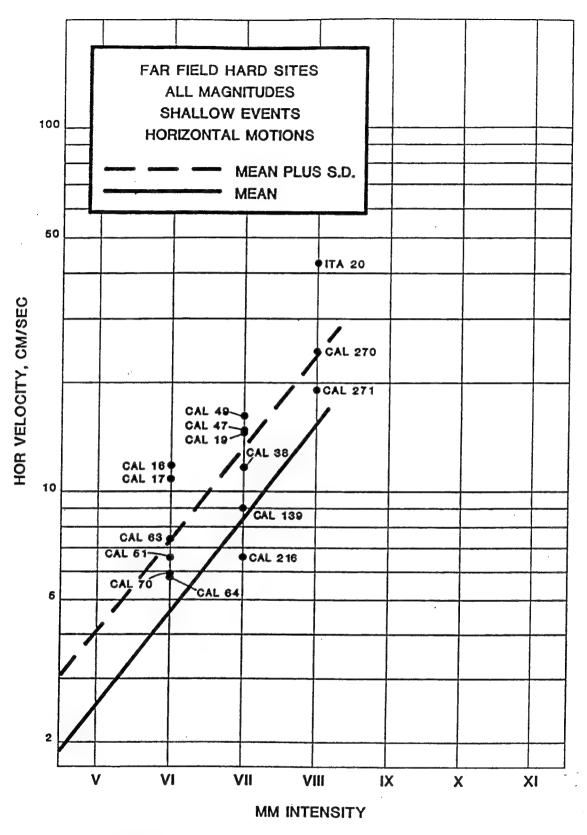


Figure 8. Accelerograms for velocity and intensity for shallow earthquakes at far-field hard sites.

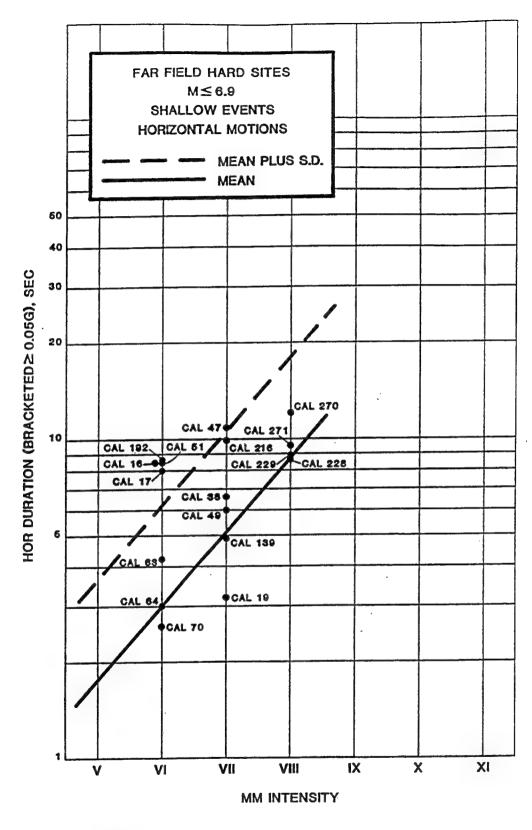


Figure 9. Accelerograms for duration and intensity for shallow earthquakes at far-field hard sites.

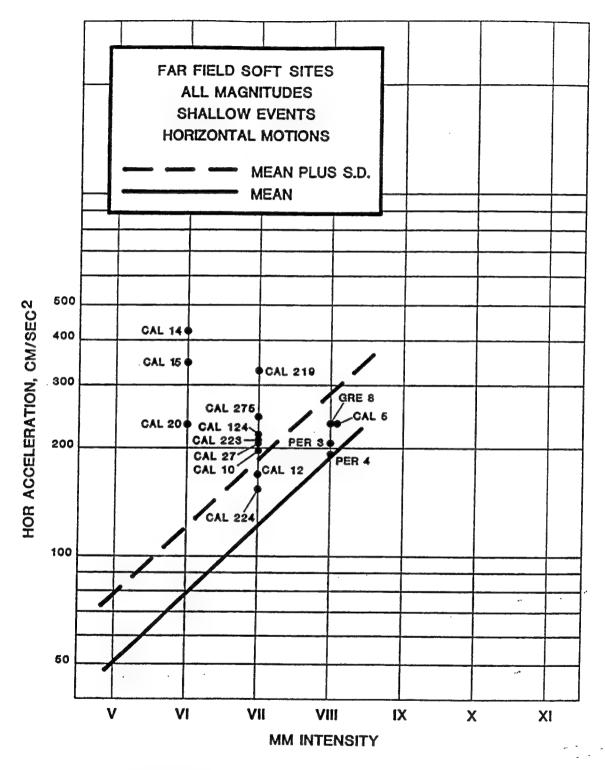


Figure 10. Accelerograms for acceleration and MM intensity for shallow earthquakes at far-field soft sites.

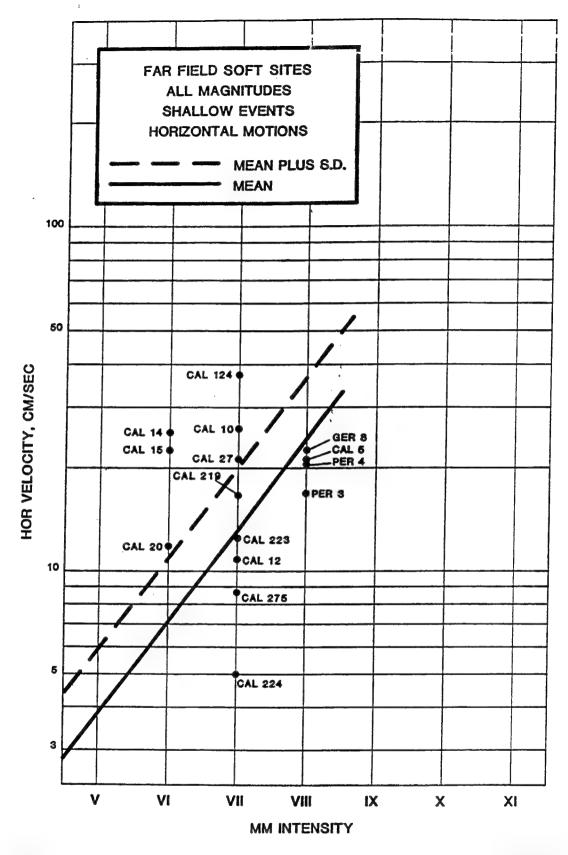


Figure 11. Accelerograms for velocity and intensity for shallow earthquakes at far-field soft sites.

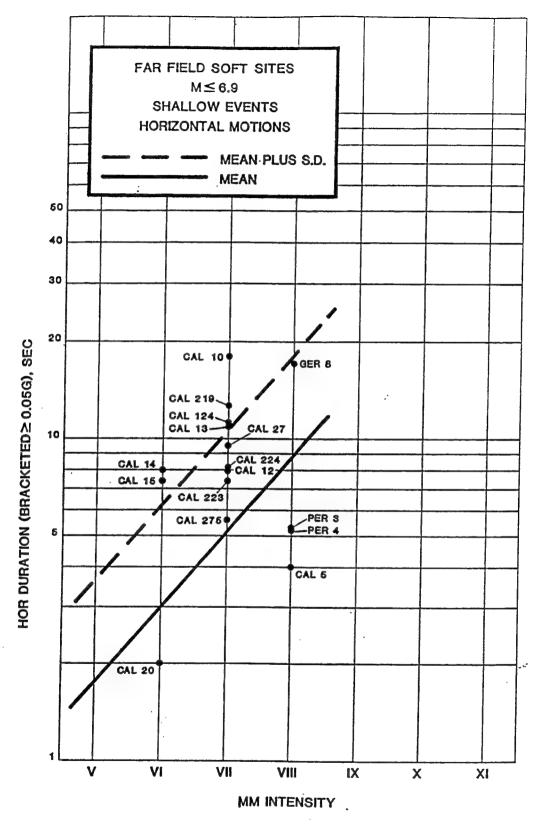


Figure 12. Accelerograms for duration and intensity for shallow earthquakes, $M \ge 6.9$ at far-field soft sites.

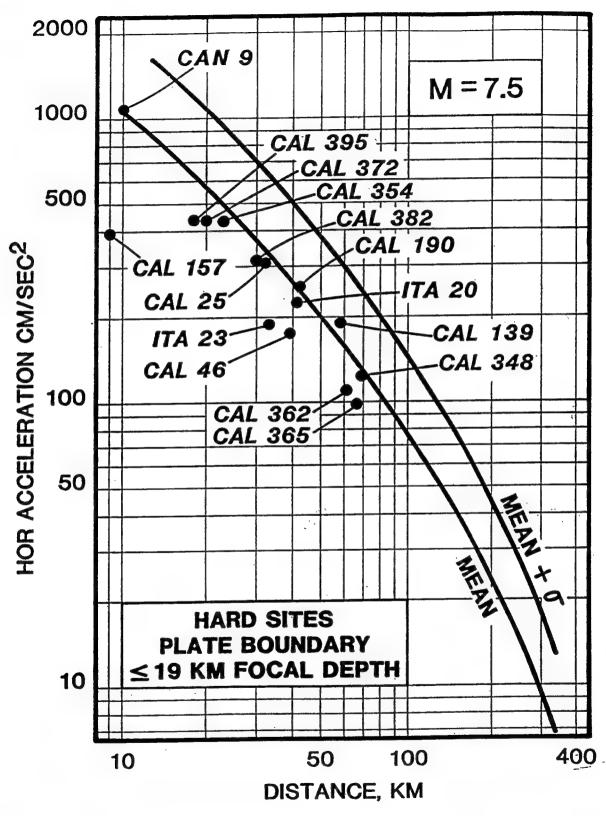


Figure 13. Accelerograms for acceleration, M = 7.5, and distance from source for shallow earthquakes at hard sites.

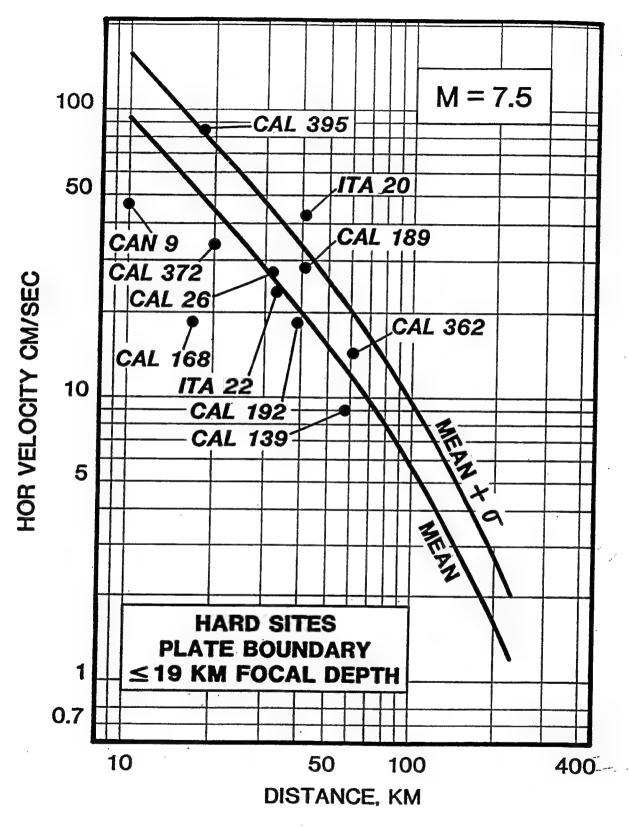


Figure 14. Accelerograms for velocity, M = 7.5, and distance from source for shallow earthquakes at hard sites.

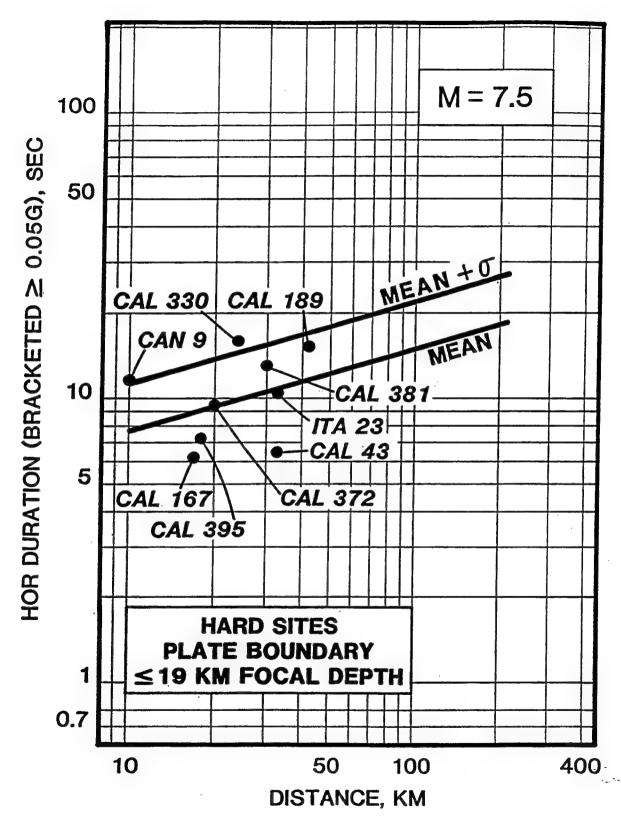


Figure 15. Accelerograms for duration, M = 7.5, and distance from source for shallow earthquakes at hard sites.

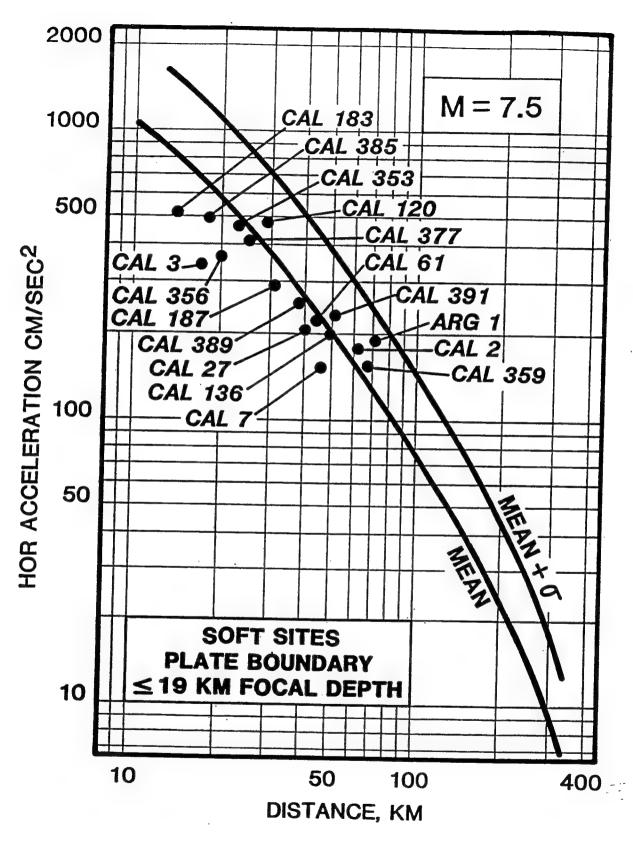


Figure 16. Accelerograms for acceleration, M = 7.5, and distance from source for shallow earthquakes at soft sites.

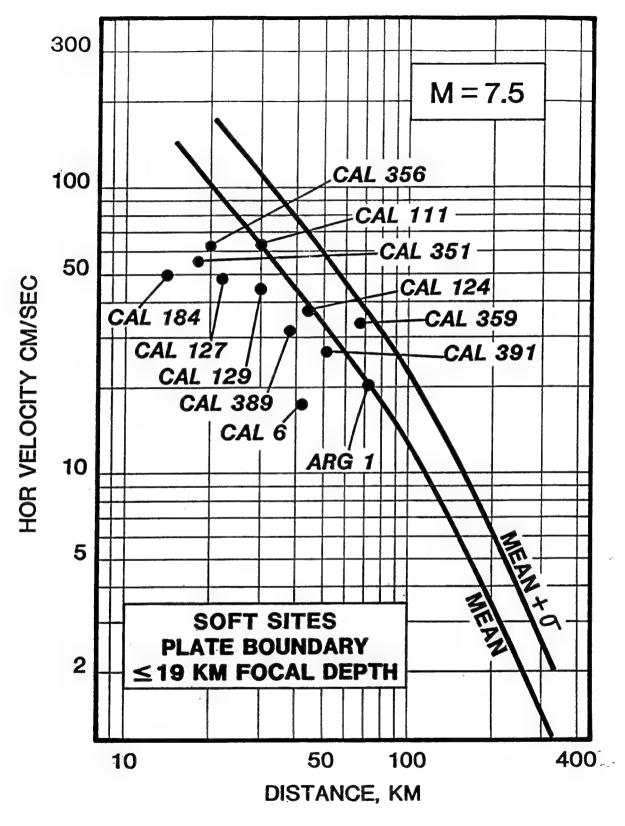


Figure 17. Accelerograms for velocity, M = 7.5, and distance from source for shallow earthquakes at soft sites.

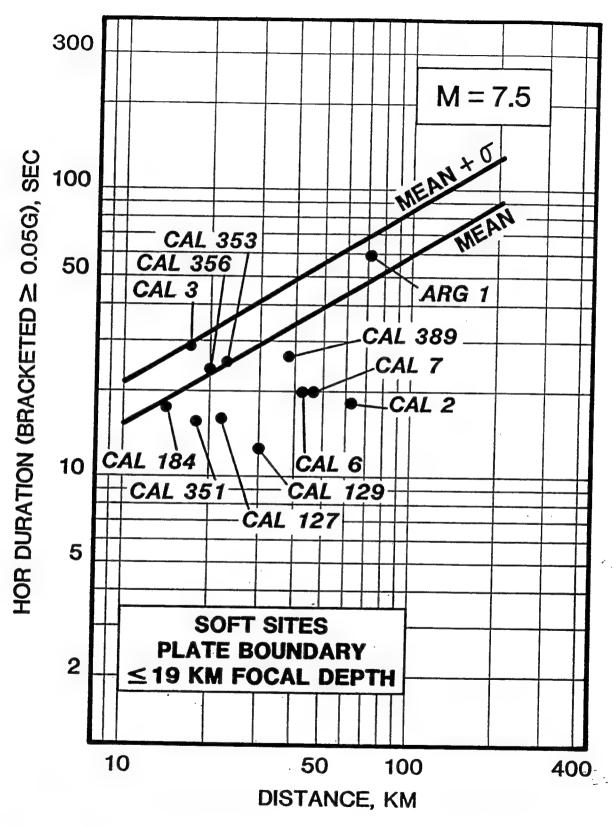
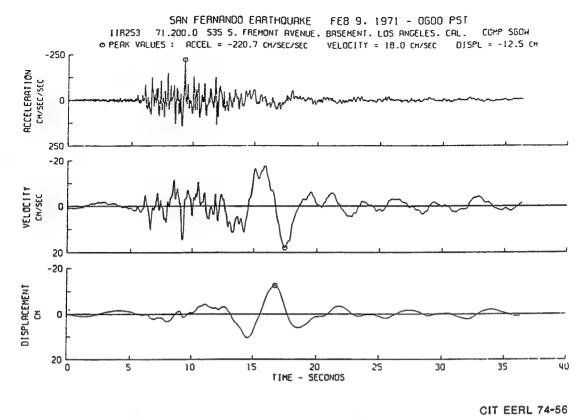
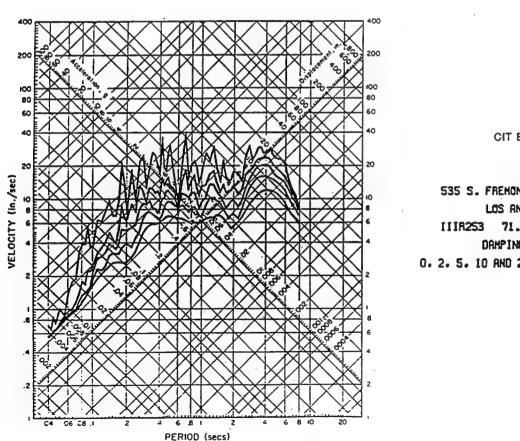


Figure 18. Accelerograms for duration, M = 7.5, and distance from source for shallow earthquakes at soft sites.

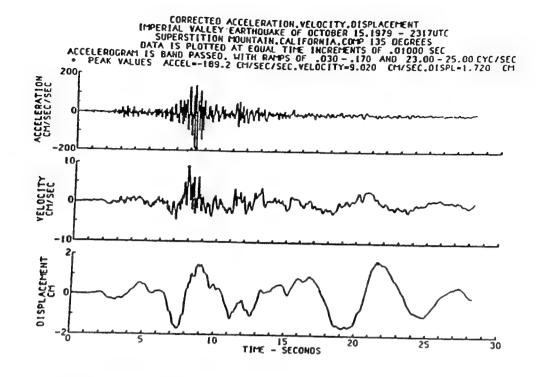


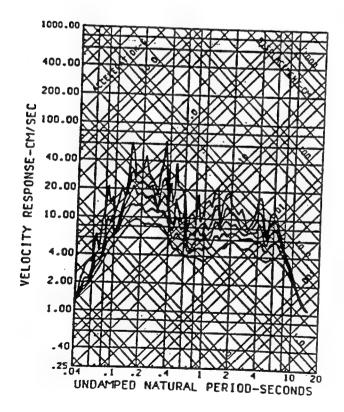


CIT EERL 74-85

S35 S. FREMONT AVENUE. BRSEMENT
LOS ANGELES. CAL.
IIIR253 71.200.0 COMP S60H
DAMPING VALUES ARE
0. 2. 5. 10 AND 20 PERCENT OF CRITICAL

Figure 19. San Fernando earthquake Feb 9, 1971 - 0600 PST, CAL 61.





USGS OF 80-703

SEISMIC ENGINEERING BRANCH/USGS BAND PASSED FROM

.030- .170 TO 23.00-25.00 HZ 2317.135DEG CRITICAL DAMPING 0.2.5.10.20 PERCENT

Figure 20. Superstition MT. CAL. 10/15/79, CAL 139.

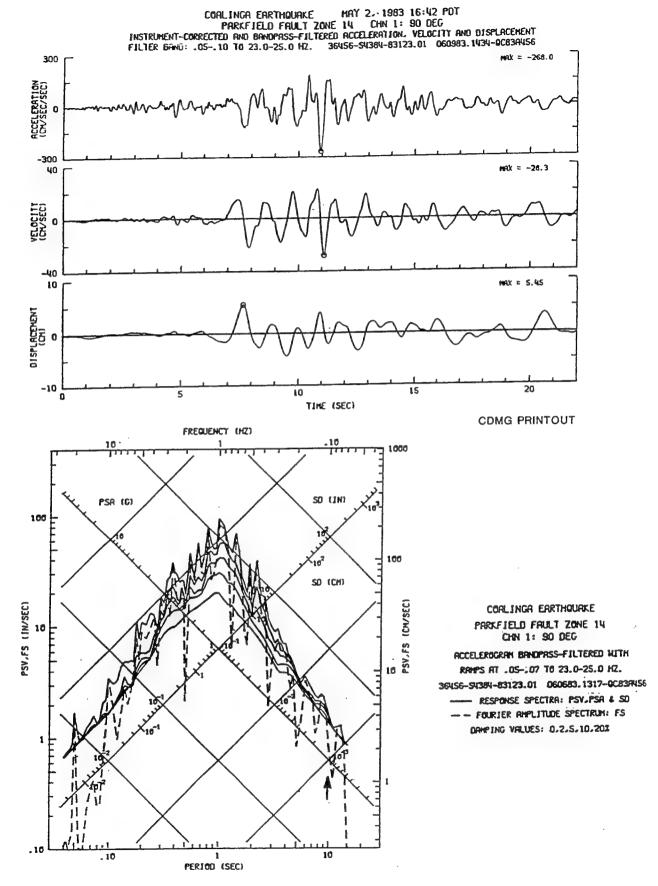


Figure 21. Coalinga Earthquake, Parkfield fault Zone 14, CHN 1: 90 Deg May 2, 1983 16:42 PDT, CAL 189.

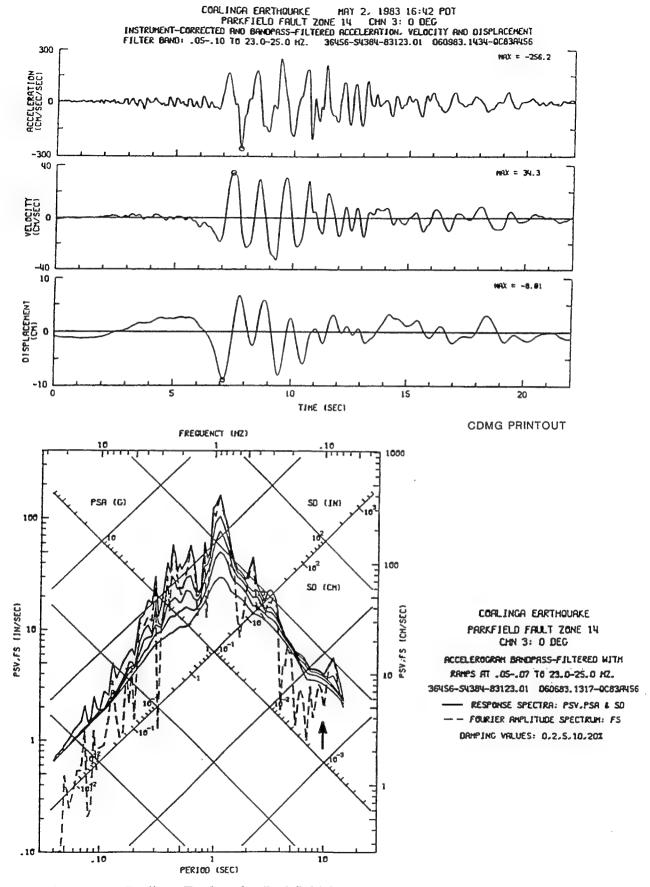
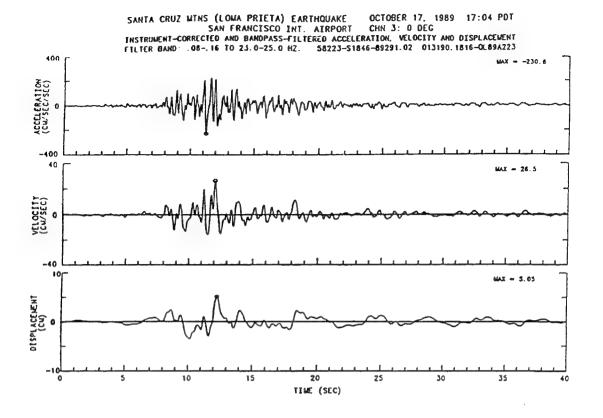


Figure 22. Coalinga Earthquake, Parkfield fault Zone 14, CHN 3: 0 Deg May 2, 1983, 16:42 PDT, CAL 190.



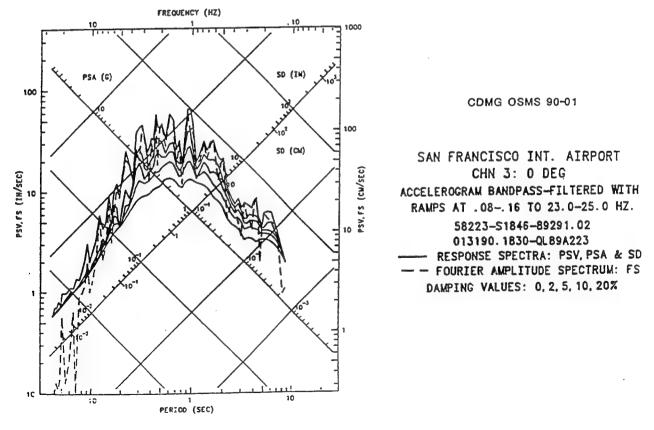


Figure 23. Santa Cruz Mtns (Loma Prieta) Earthquake, Oct 17, 1989, 17: 04 PDT, CAL 391.

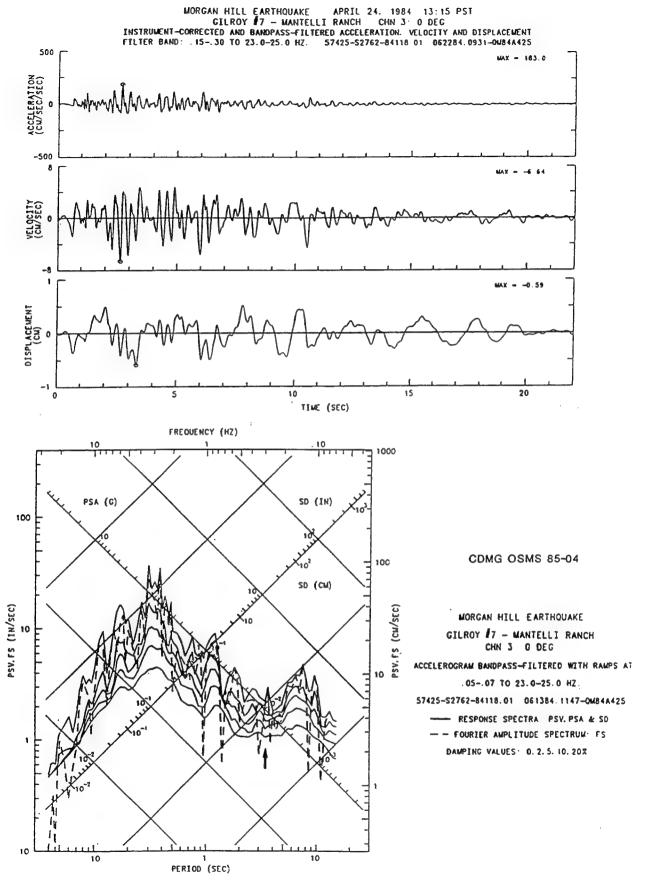
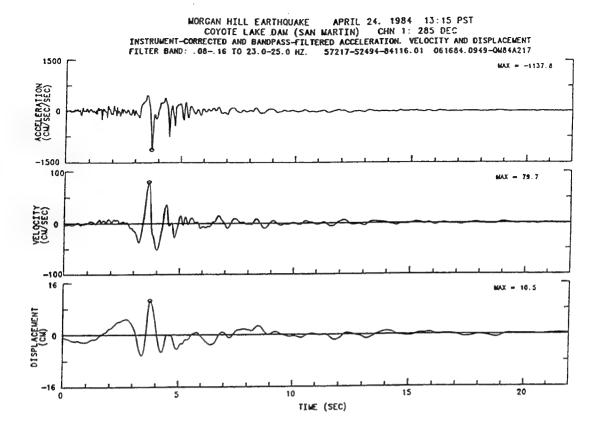


Figure 24. Morgan Hill Earthquake, Gilroy No. 7 - Mantelli Ranch, CHN 3, 0 Deg, April 24, 1984, 13:15 PST, CAL 216.



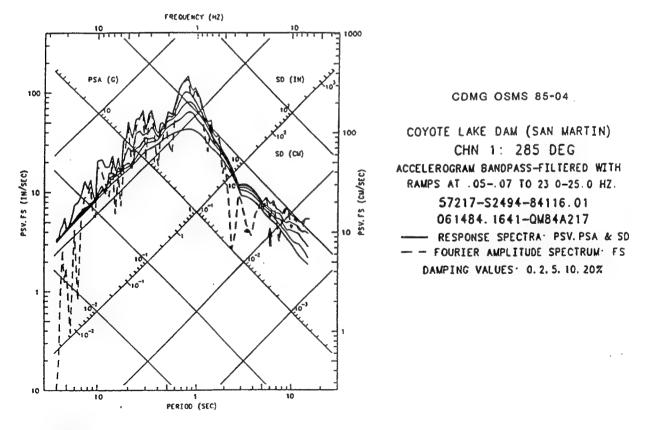


Figure 25. Morgan Hill Earthquake, April 24, 1984, 13:15 PST, CAL 228.

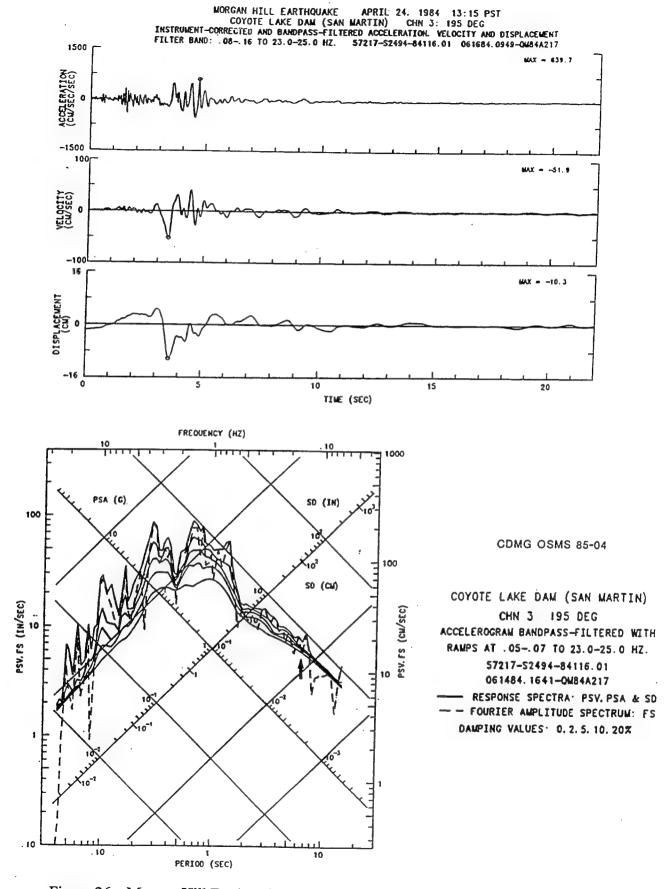
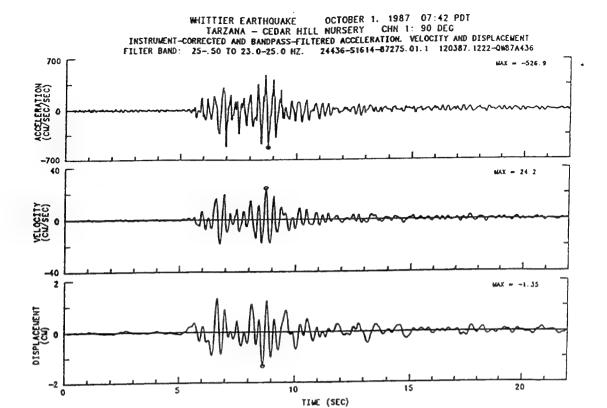


Figure 26. Morgan Hill Earthquake, April 24, 1984, 13:15 PST, CAL 229.



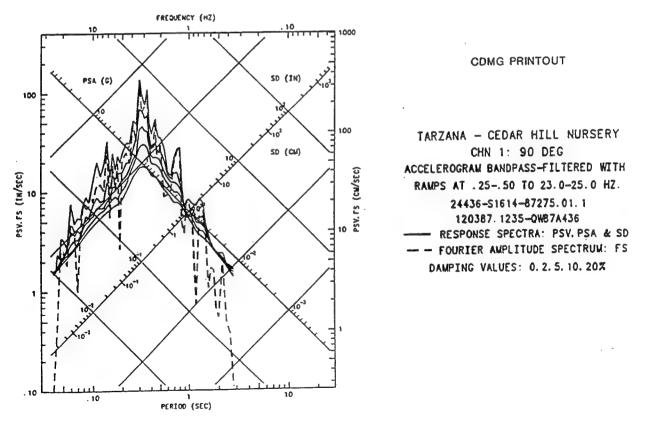
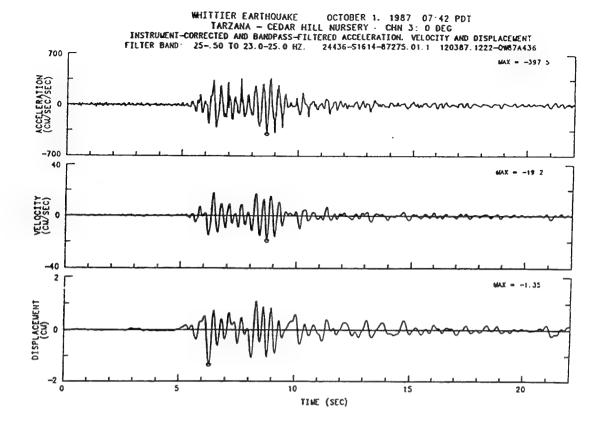


Figure 27. Whittier Earthquake, Oct 1, 1987, 07 42 PDT, CAL 270.



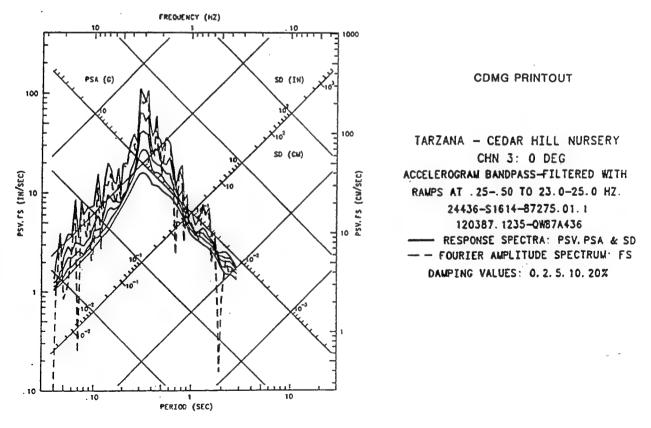
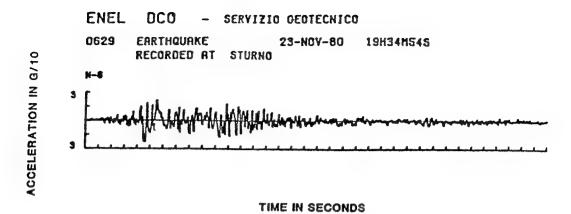


Figure 28. Whittier Earthquake, Oct 1, 1987, 07 42 PDT, CAL 271.



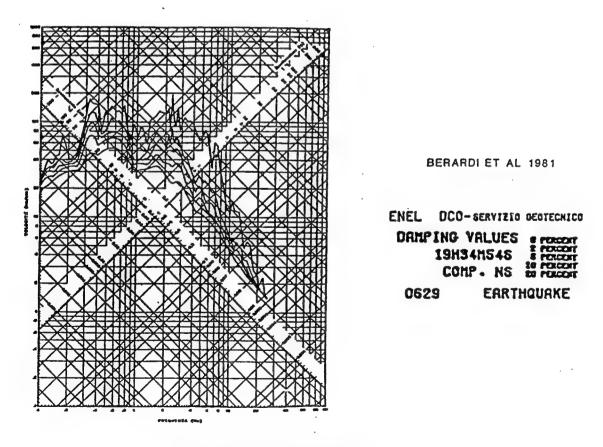
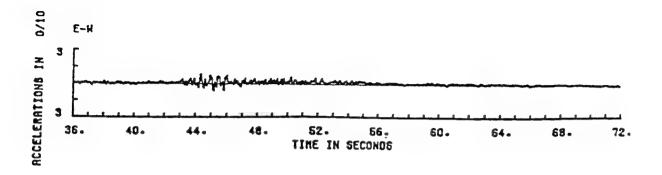


Figure 29. Sturno, Italy, ITA 20.

ENEL DCO - SERVIZIO DEDTECNICO

0629 ERRTHQUAKE 23-NOV-80 19H34H54S
RECORDED AT STURNO



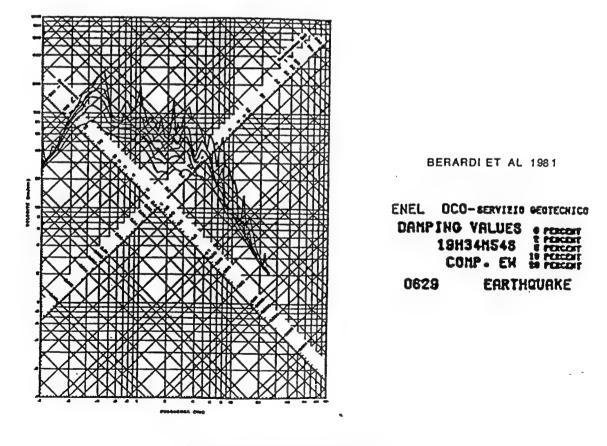


Figure 30. Sturno, Italy, ITA 21.

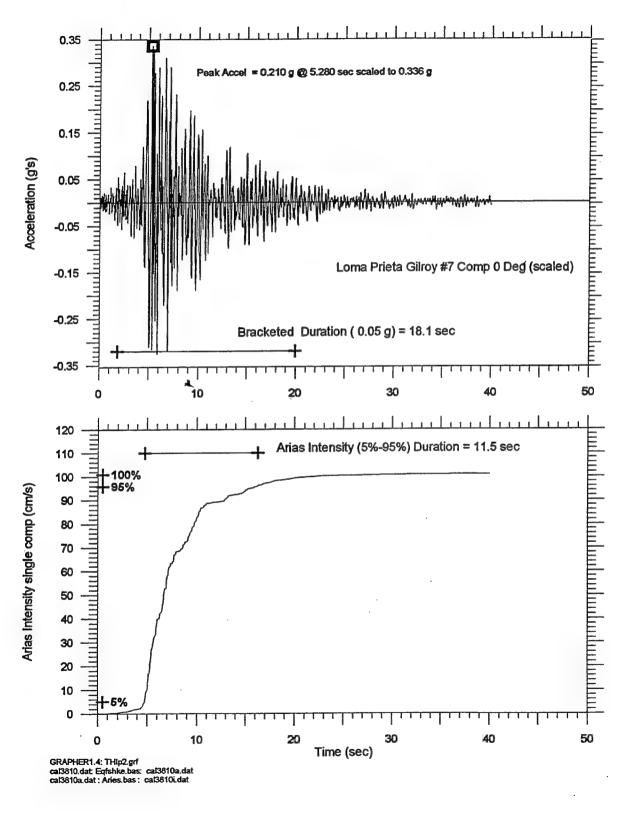


Figure 31. Loma Prieta Gilroy # 7, 0 degree component, scaled

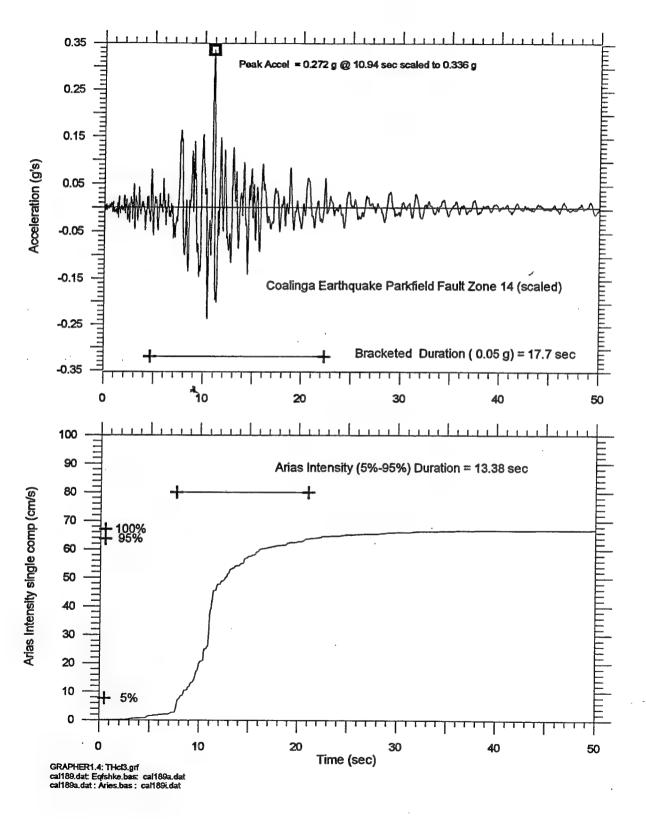


Figure 32. Coalinga earthquake, Parkfield Fault Zone 14, scaled

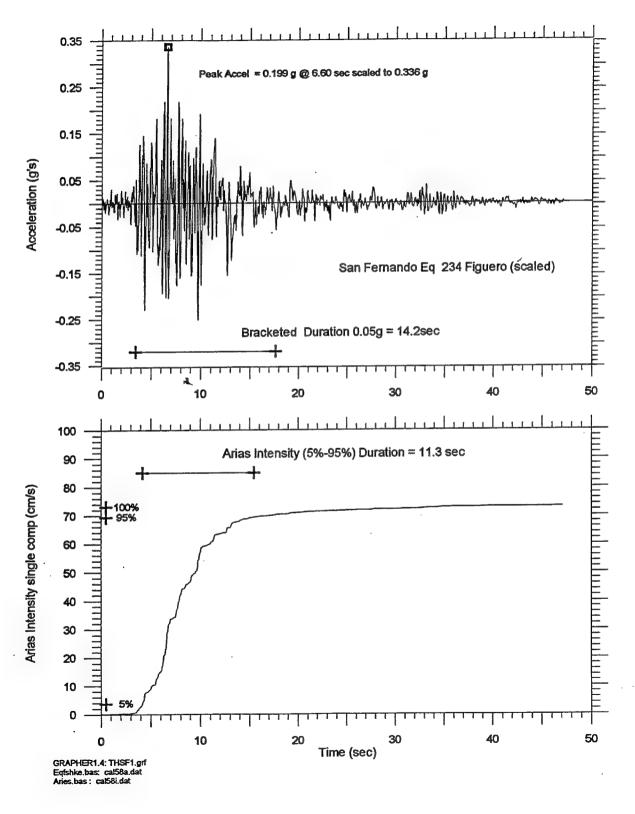


Figure 33. San Fernando earthquake, 234 Figuero, scaled

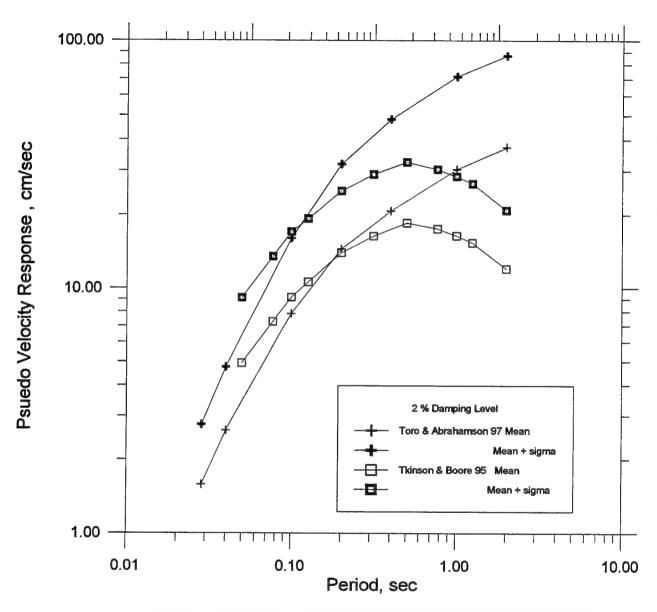


Figure 34a. Psuedo velocity response spectrum for 2 % damping for the Toro & Abrahamson and the Atkinson & Boore attenution relationships.

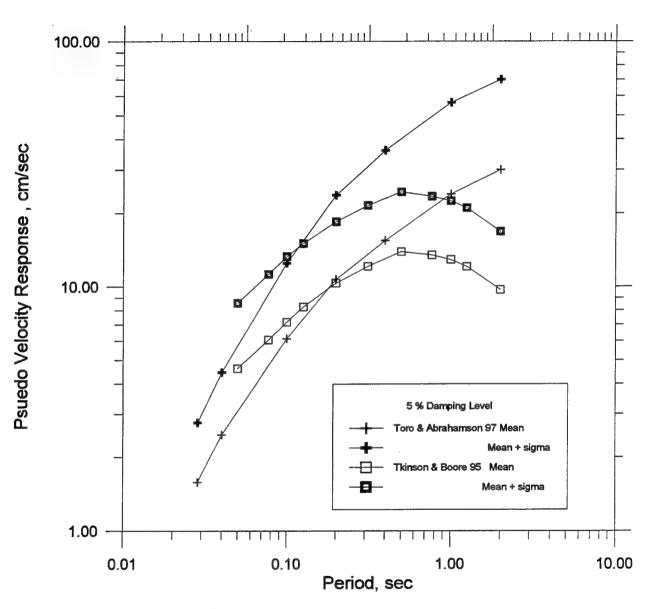


Figure 34b. Psuedo velocity response spectrum for 5 % damping for the Toro & Abrahamson and the Atkinson & Boore attenution relationships.

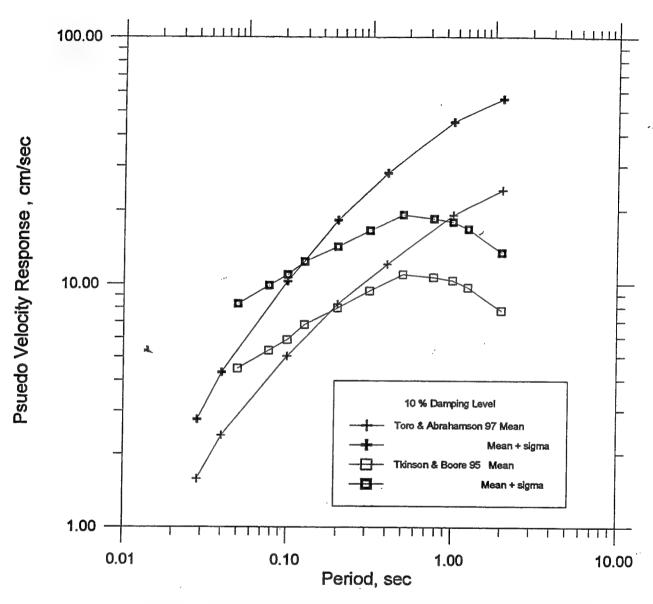


Figure 34c. Psuedo velocity response spectrum for 10 % damping for the Toro & Abrahamson and the Atkinson & Boore attenution relationships.

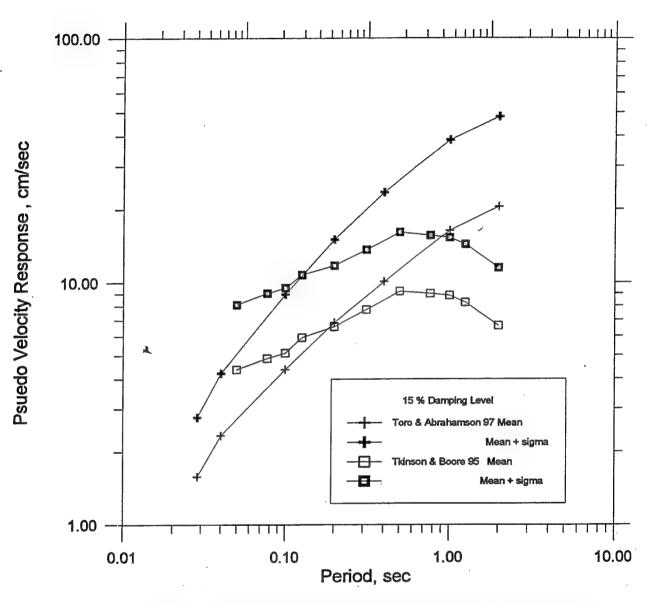


Figure 34d. Psuedo velocity response spectrum for 15 % damping for the Toro & Abrahamson and the Atkinson & Boore attenution relationships.

Stephen Powerhouse Design Earthquake Response Spectra Comparison

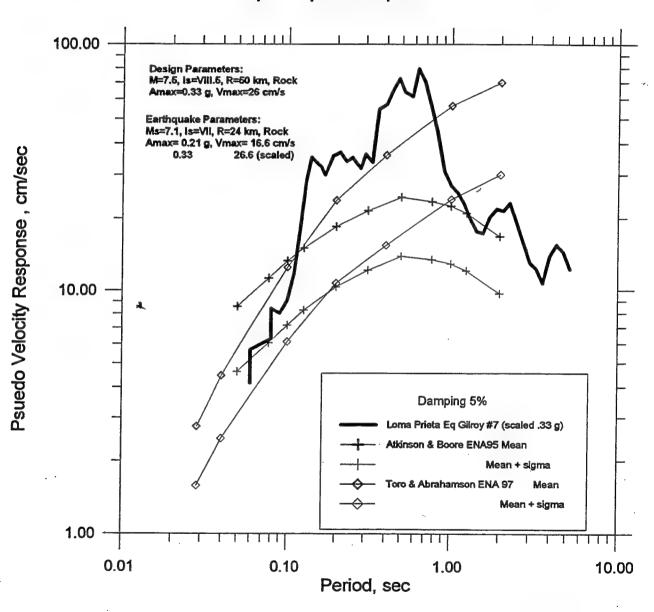


Figure 35. MCE and Loma Prieta Gilroy # 7 response spectra (5 % damping)

Stephen: Powerhouse Design Earthquake Response Spectra Comparison

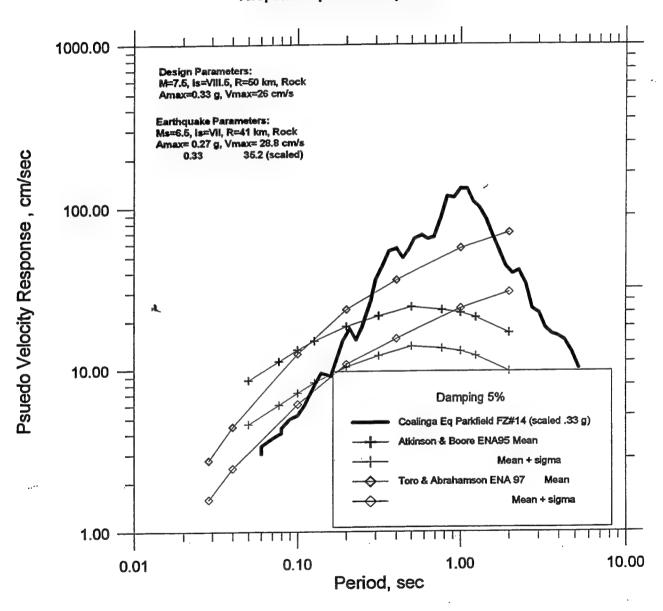


Figure 36. MCE and Coalinga, Fault Zone 14 response spectra (5 % damping)

Stephen: Powerhouse Design Earthquake Response Spectra Comparison

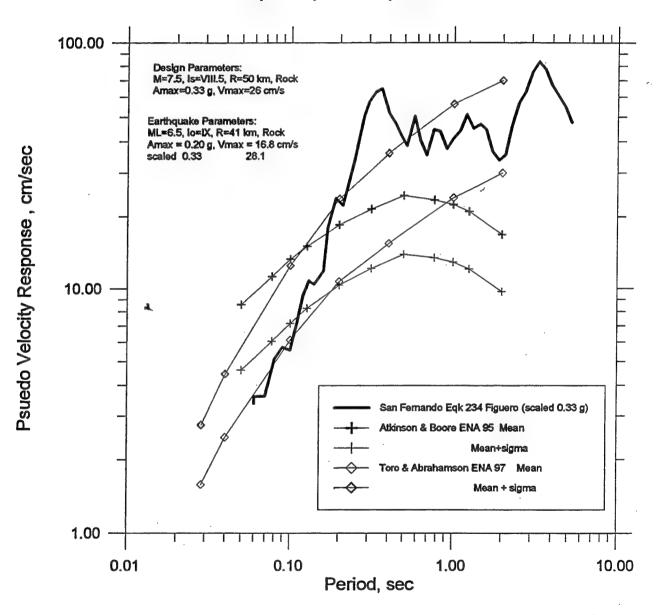
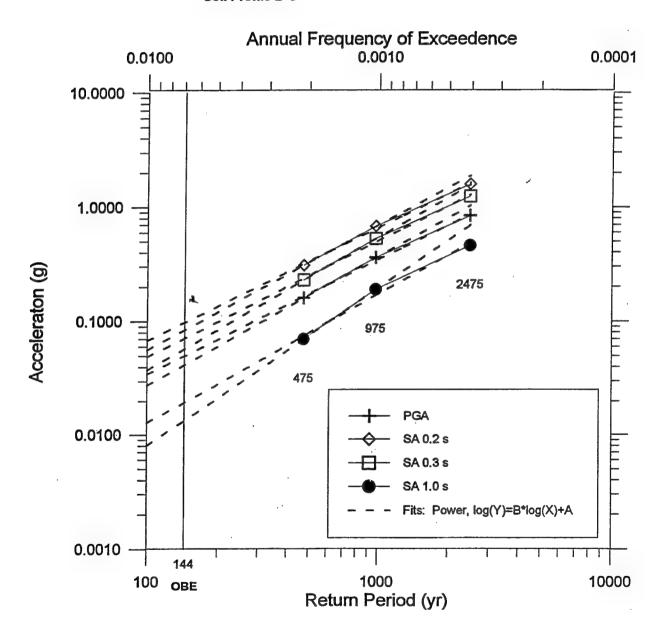


Figure 37. MCE and San Fernando earthquake, 234 Figuero response spectra (5 % damping)

Probabilistic Seismic Hazard Curve St. Stephen Powerhouse, Cooper River Diversion Project, GA NEHRP National Hazard Maps November 1996 Soil Profile B-C



GRAPHER1.4: sphprob.grf probhz1.dat

Figure 38. USGS Probabilistic seismic hazard curves for St. Stephen Powerhouse site

Charleston, SC Probabilistic Seismic Hazard Return Period 2475 yrs PGA

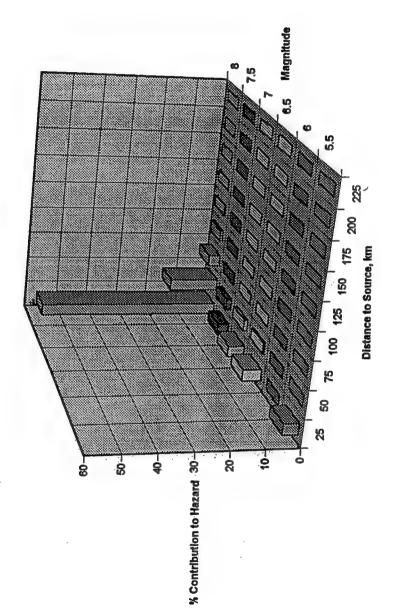


Figure 39. Deaggregated PGA hazard, Charleston, South Carolina

Charleston,SC Probabilitic Seismic Hazard Return Period 2475 yrs SA=1 Hz

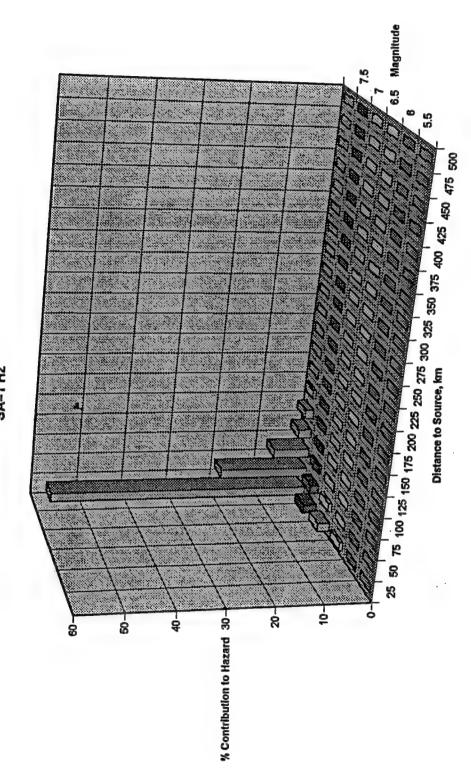


Figure 40. Deaggregated SA(1 Hz) hazard, Charleston, South Carolina

Charleston, SC
Probabilistic Seismic Hazard
Return Period 2475 yrs
SA=3.3 Hz

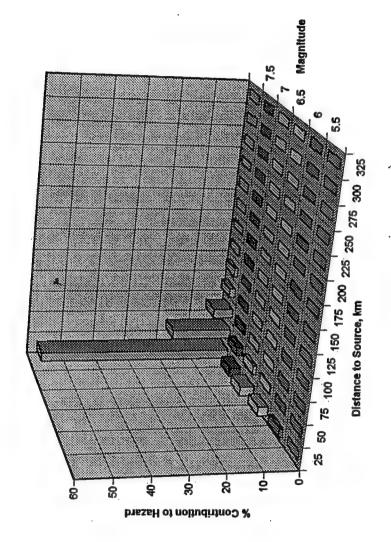


Figure 41. Deaggregated SA(3.3 Hz) hazard, Charleston, South Carolina

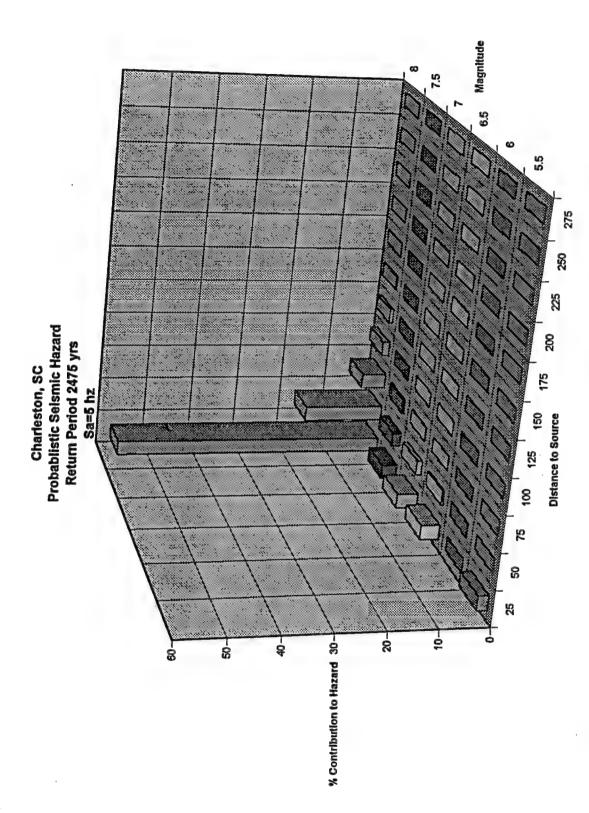


Figure 42. Deaggregated SA(5 Hz) hazard, Charleston, South Carolina

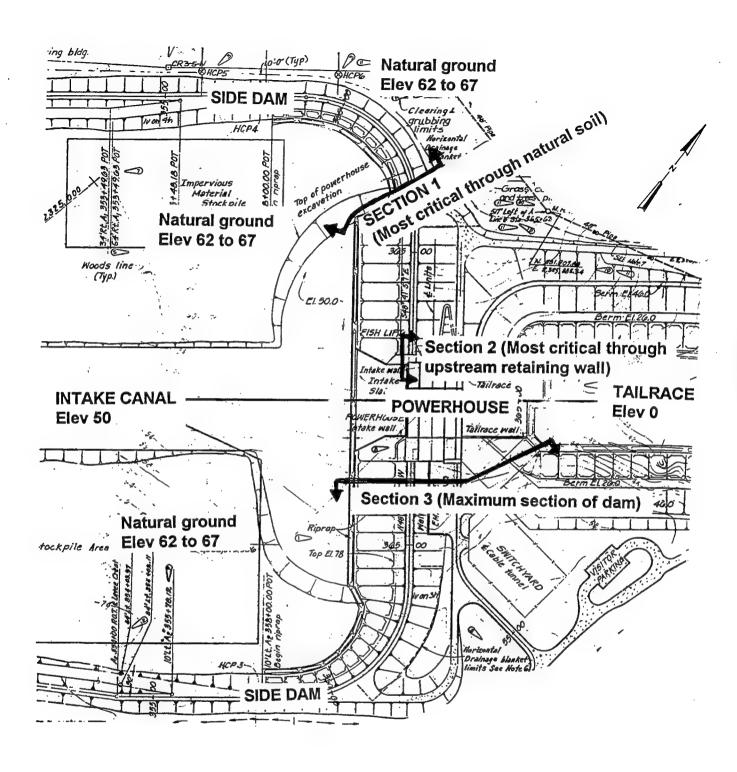


Figure 43. Plan of St. Stephen Powerhouse Project

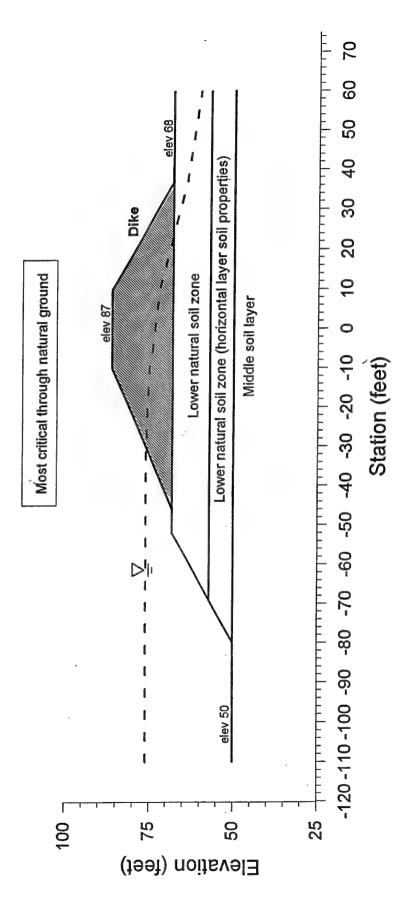


Figure 44. Section 1, as idealized, embankment on natural foundation deposit

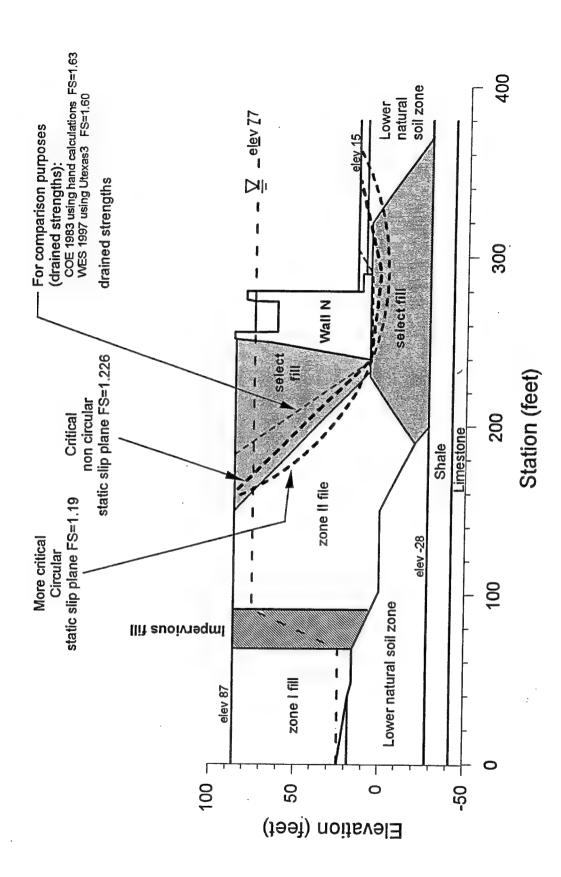


Figure 45. Section 2, as idealized, upstream retaining wall

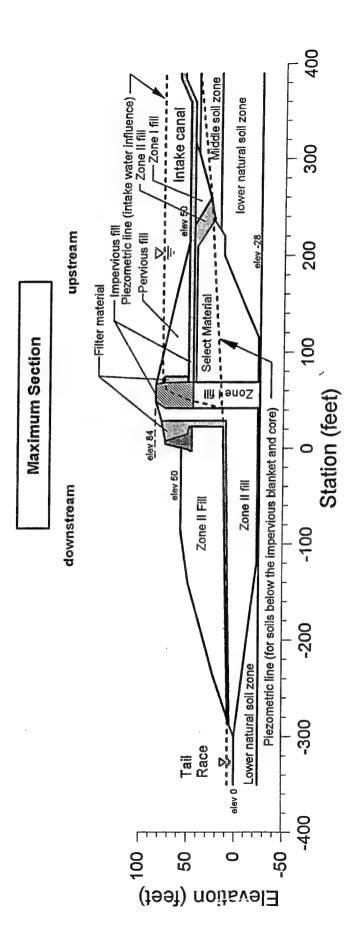


Figure 46. Section 3, as idealized, maximum section of embankment dam

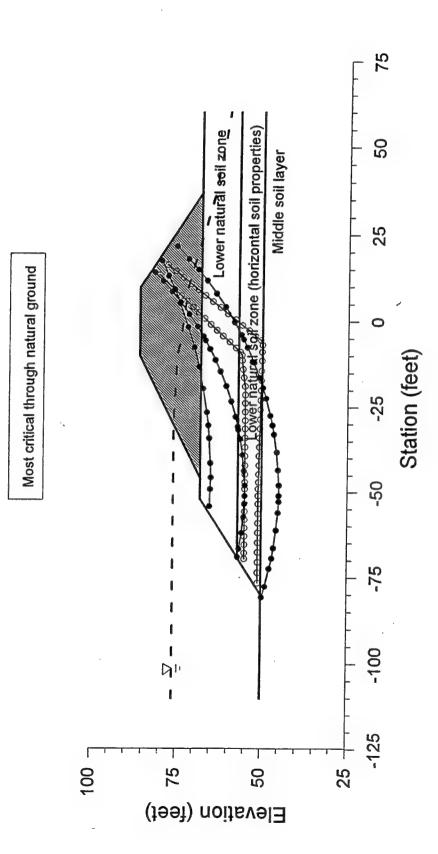


Figure 47. Yield acceleration slip surfaces, Section 1

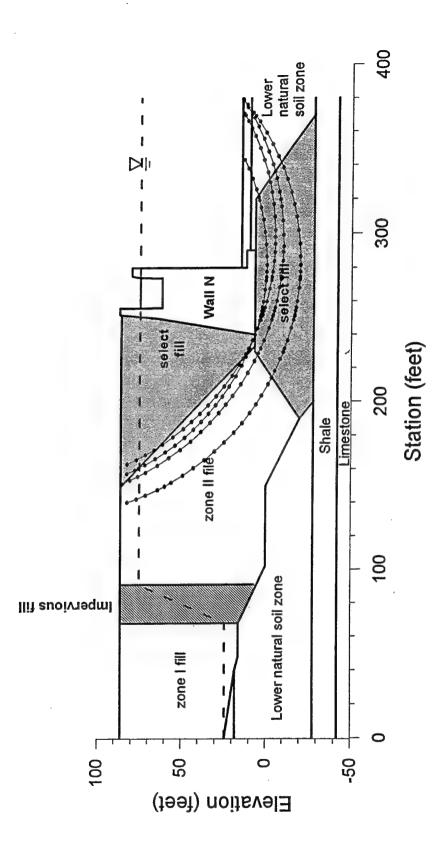


Figure 48. Yield acceleration slip surfaces, Section 2

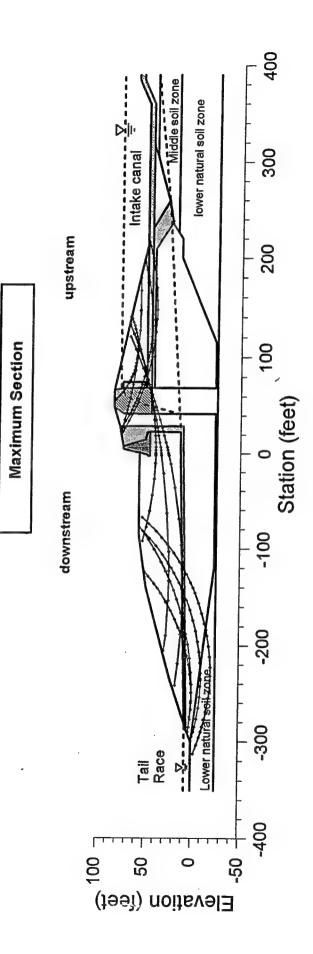


Figure 49. Yield acceleration slip surfaces, Section 3

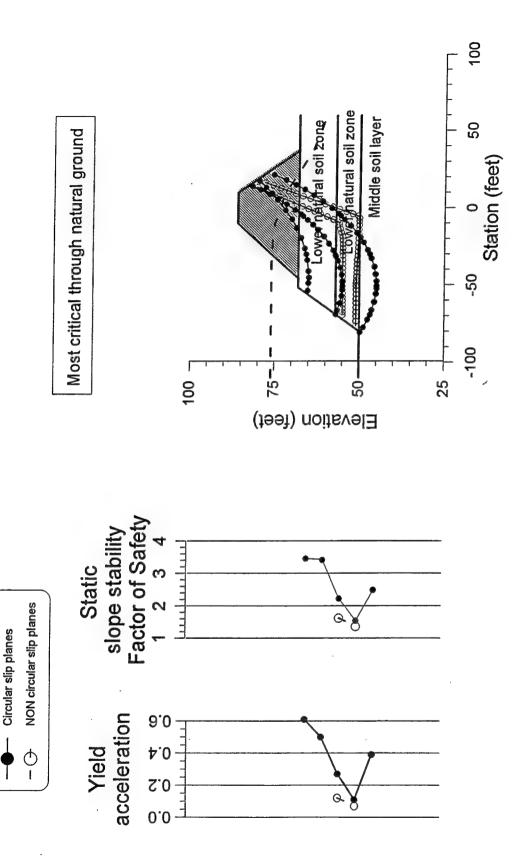


Figure 50. Section 1 yield accelerations, static factors of safety against sliding

Critical section through upstream retaining walls

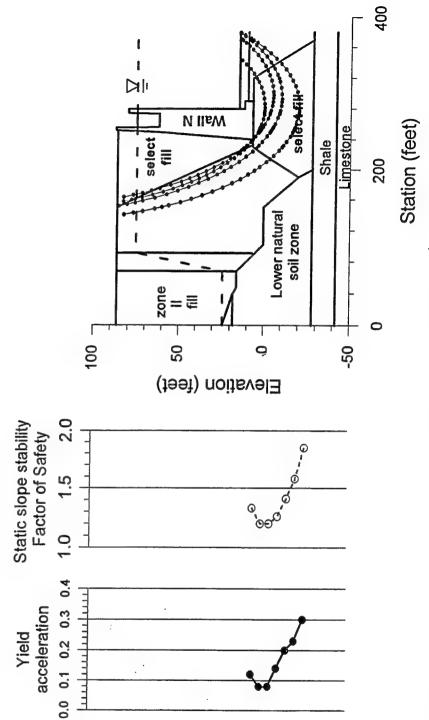


Figure 51. Section 2 yield accelerations, static factors of safety against sliding

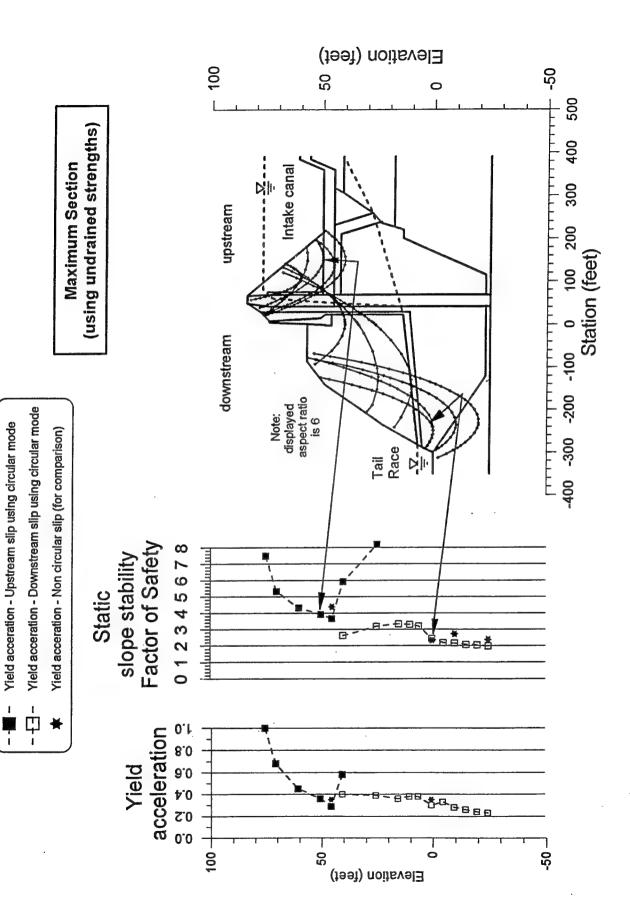


Figure 52. Section 3 yield accelerations, static factors of safety against sliding

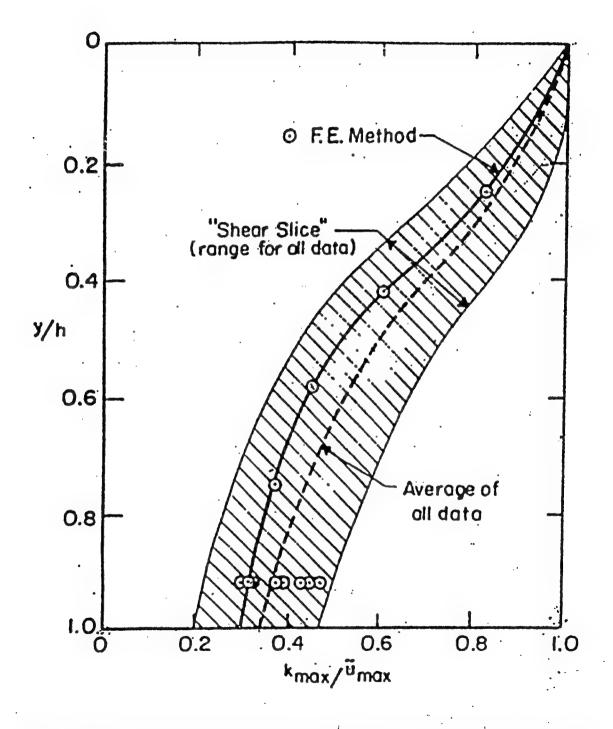


Figure 53. Makdisi-Seed dynamic response chart for Newmark-type deformation analysis (after Makdisi and Seed 1977)

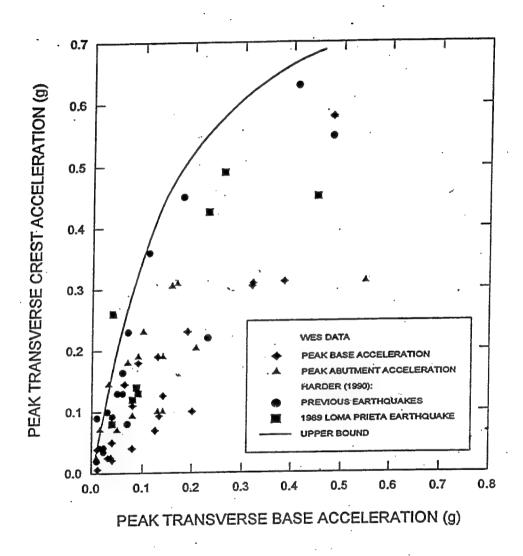


Figure 54. Upper-bound relationship between crest and base or abutment response for dams (after Harder 1991, as modified by WES 1996)

Maximum Section

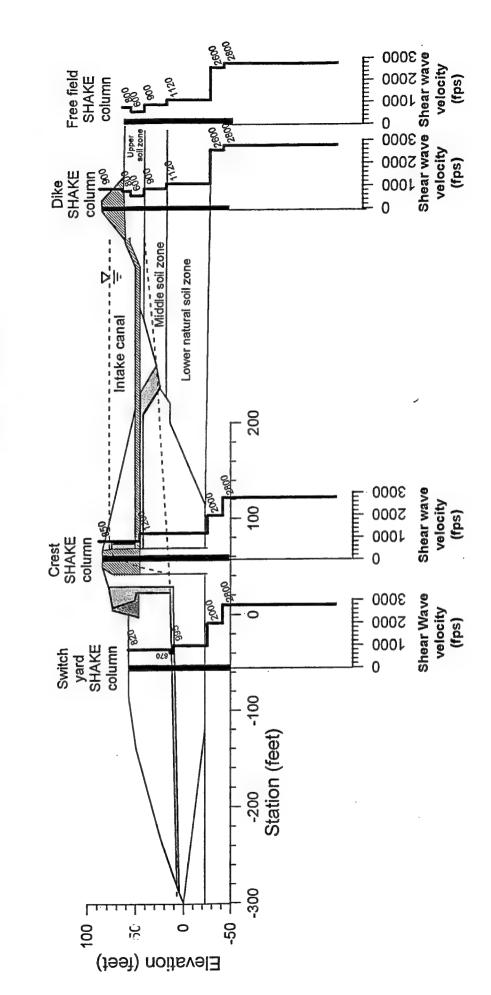
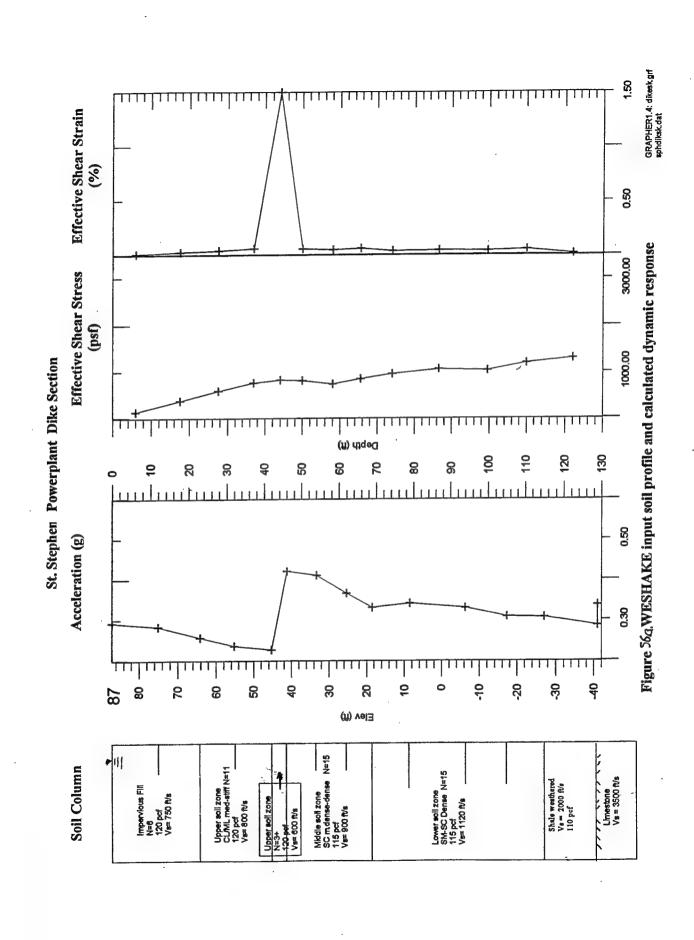
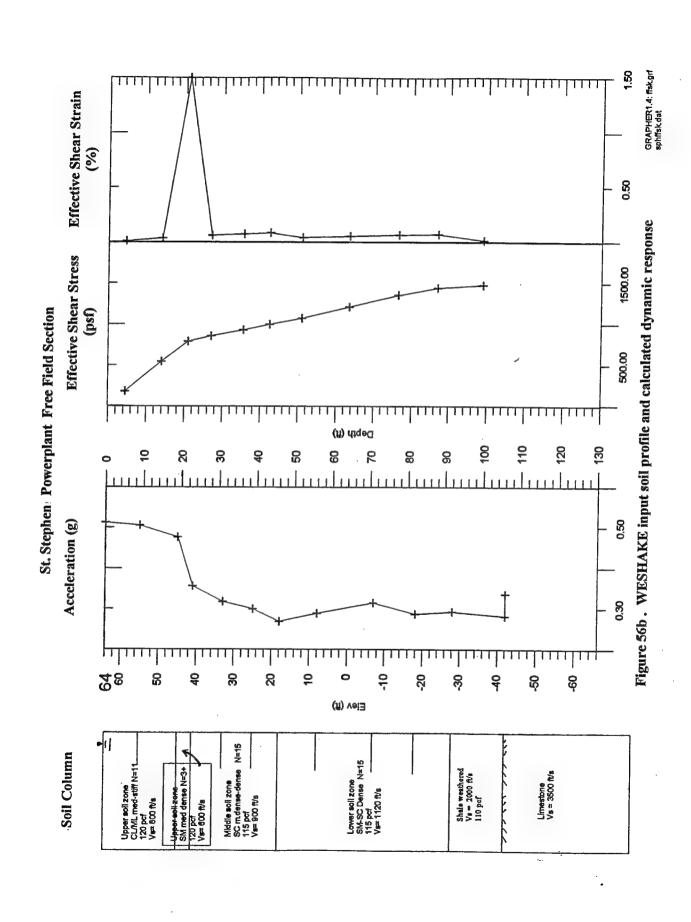
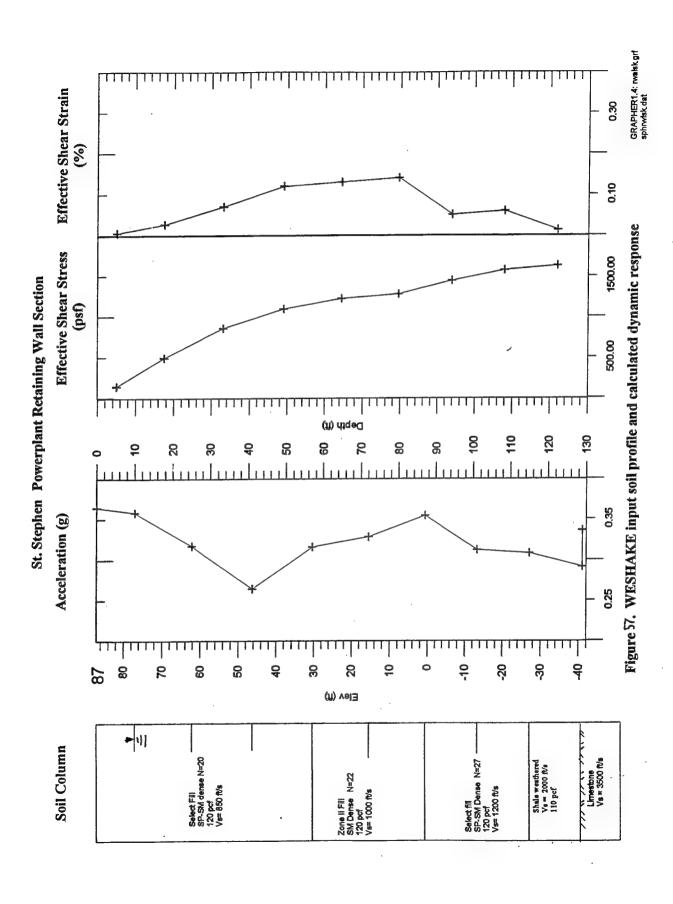
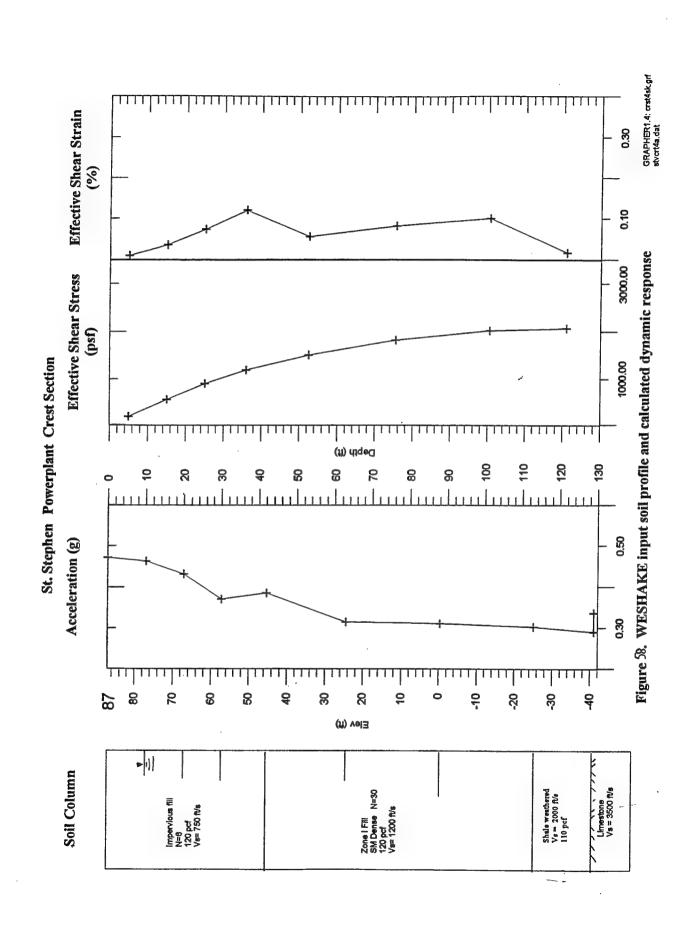


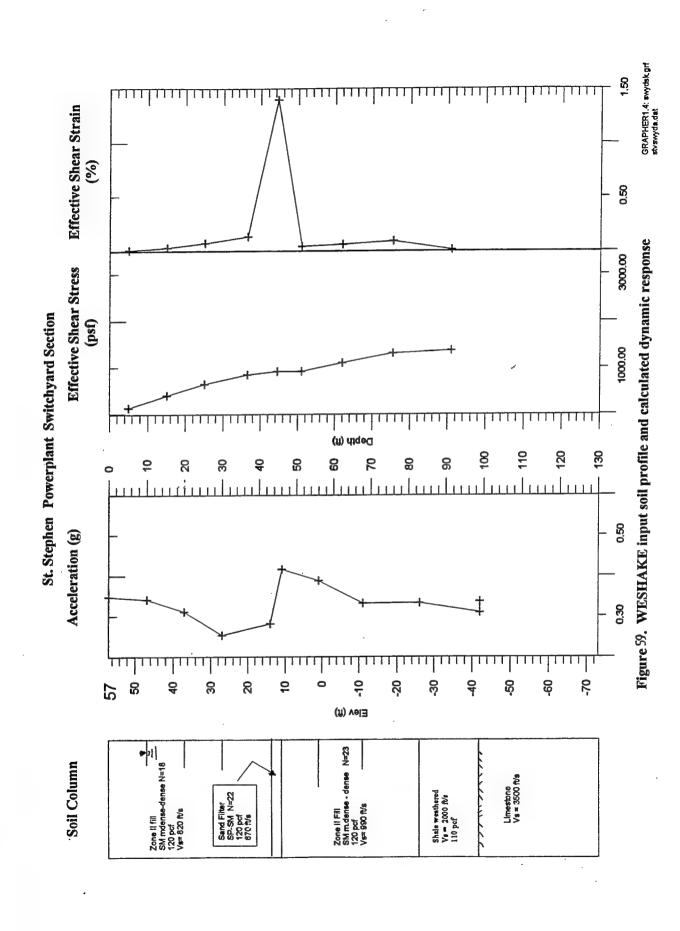
Figure 55. Locations of SHAKE profiles











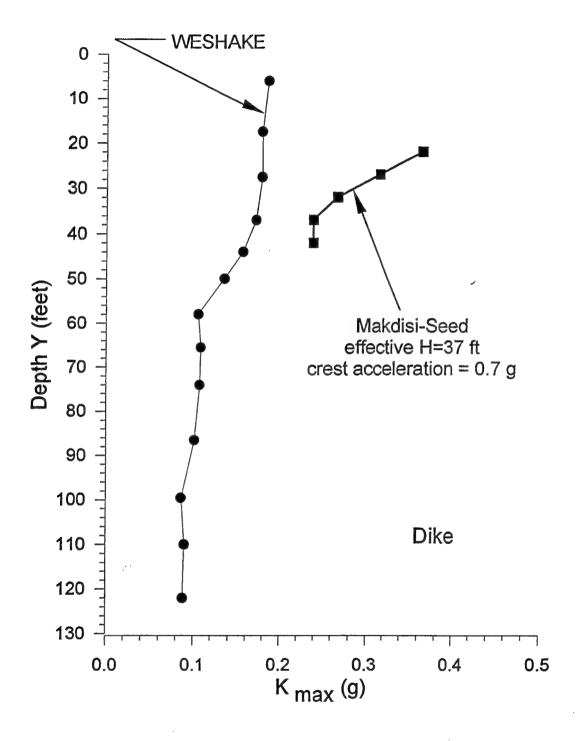


Figure 60. k_{max} values for Section 1

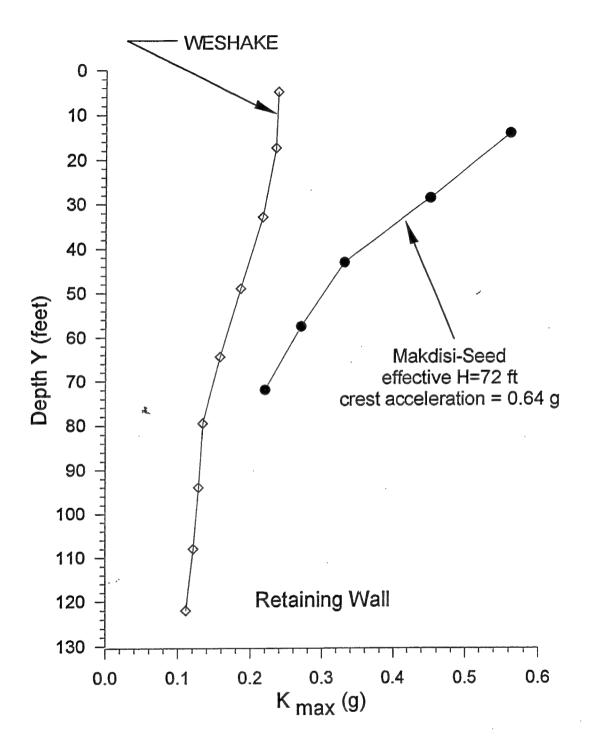


Figure 61. k_{max} values for Section 2

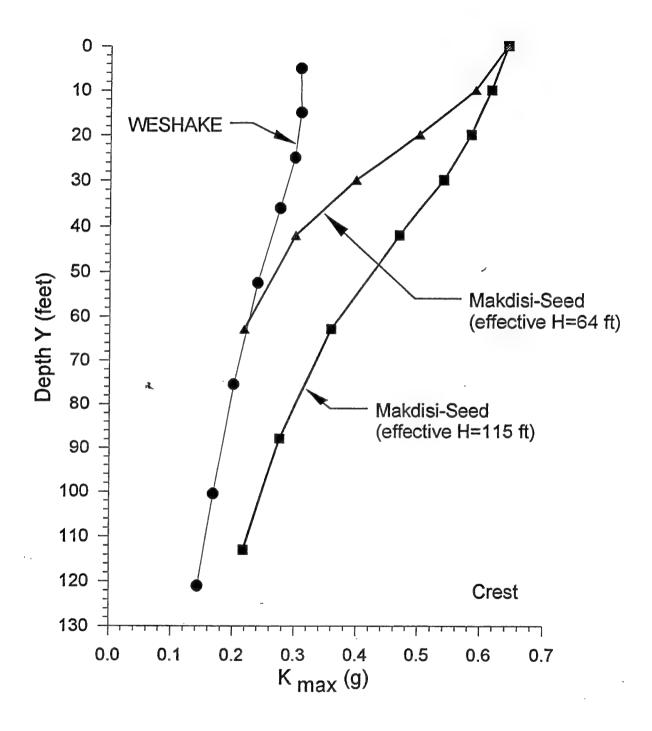


Figure 62. k_{max} values for Section 3, crest and upstream surfaces

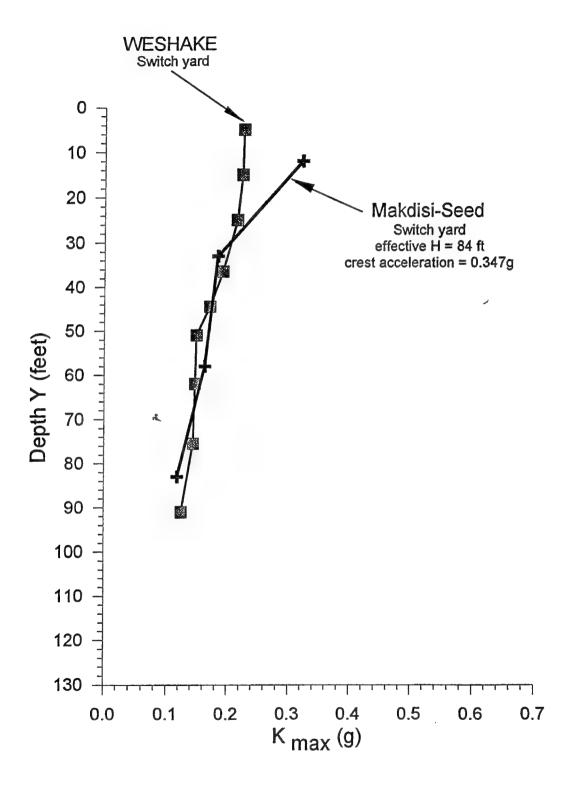


Figure 63. k_{max} values for Section 3, switchyard and downstream surfaces

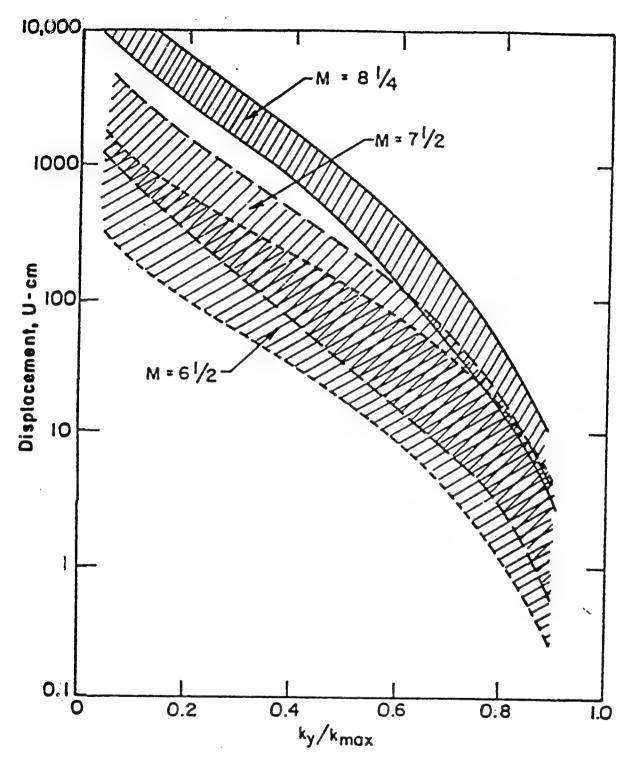


Figure 64. Makdisi-Seed displacement chart (after Makdisi and Seed 1977).

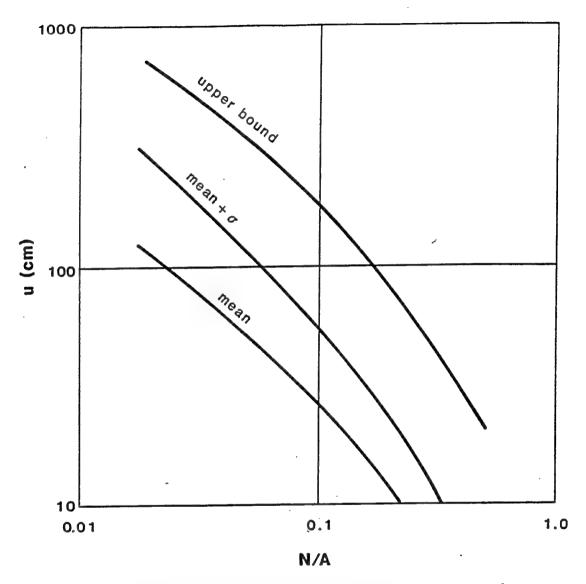


Figure 65. Hynes-Franklin displacement chart (Note: $N=k_{yield}$, $A=k_{max}$, after Hynes-Griffin and Franklin 1984)

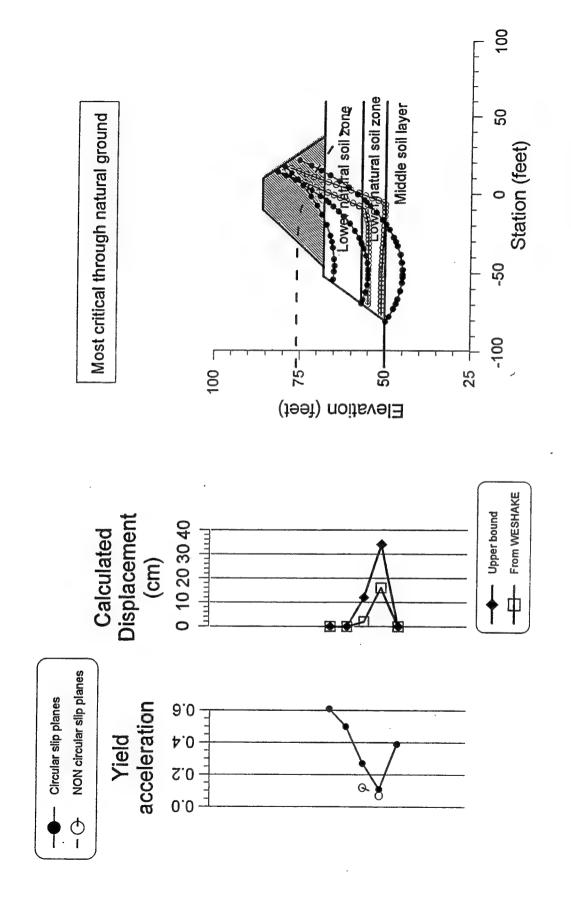


Figure 66. Displacements computed for Section 1

Critical section through upstream retaining walls

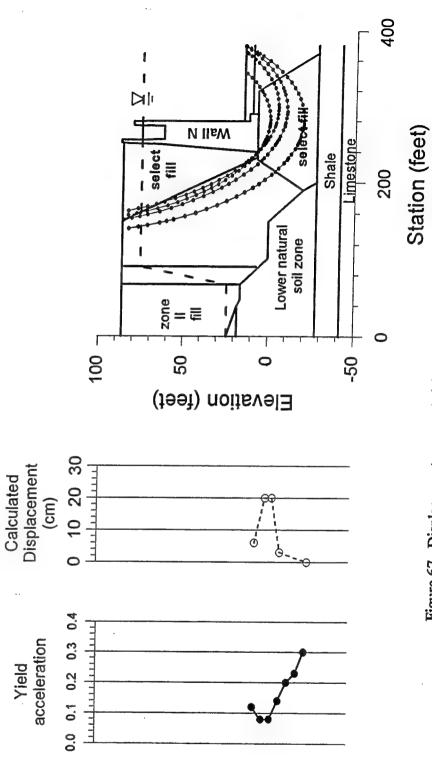


Figure 67. Displacements computed for Section 2

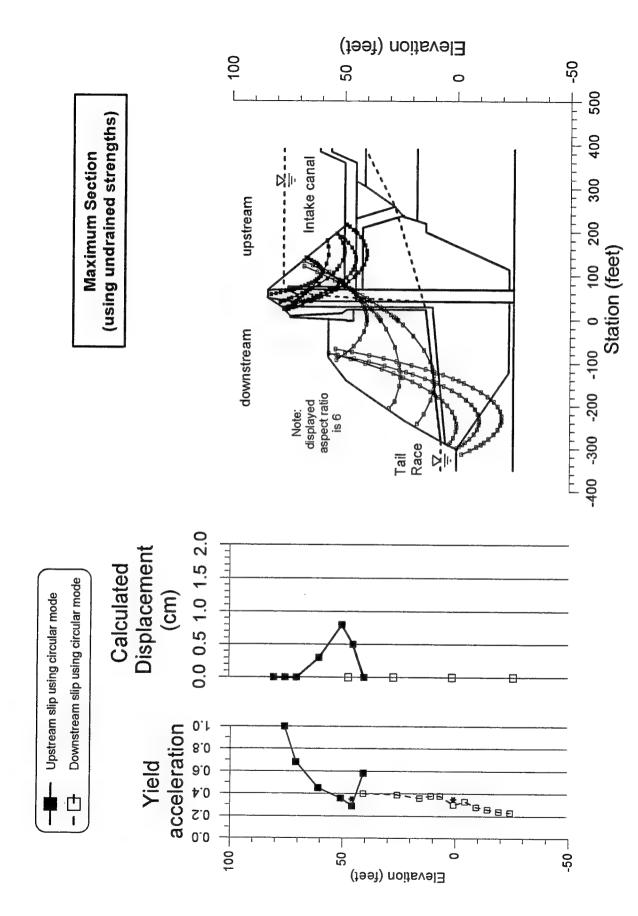


Figure 68. Displacements computed for Section 3.

APPENDIX A: SEISMIC HISTORY, M ≥ 3.5, WITHIN 150 KM OF THE ST. STEPHEN POWERHOUSE SITE. FROM THE NATIONAL GEOPHYSICAL DATA CENTER/NOAA, BOULDER, CO

SEISMICITY (M>=3.5) WITHIN 150 KM OF 33N25' 79W56'

Radial Search

NGDC EARTHQUAKE DATA FILE

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13. ABSTRACT (Maximum 200 words)			

An evaluation of the geological-seismological hazard was conducted at the St. Stephen Powerhouse Project, which is part of the cooper River Rediversion Project in South Carolina. The project is located about 60 km north of Charleston, SC, and consists of a reinforced concrete powerhouse structure founded on rock, flanked by rolled-fill earth embankments, founded partially on rock and partially on alluvium. For the purposes of this study, the alluvium is assumed to be competent, not susceptible to liquefaction. The Maximum Credible Earthquake (MCE) is estimated to correspond to a magnitude 7.5 event, 55 km from the site, resulting in peak ground accelerations at the site of 0.32 and 0.35 g. The Operating Basis Earthquake (OBE) is estimated to correspond to about a magnitude 5 event, resulting in a peak ground acceleration of 0.04 to 0.05 g at the site. The Newmark-sliding-block analyses indicate deformations in the maximum section under the MCE will be negligible, less than 1 cm. However, deformation under retaining walls and embankments founded on natural ground may be on the order of 15 to 35 cm.

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